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NATIONAL DAM SAFETY INSPECTION PROGRAM. LAKE ALGONQUIN DAM (NDS--ETC(U)
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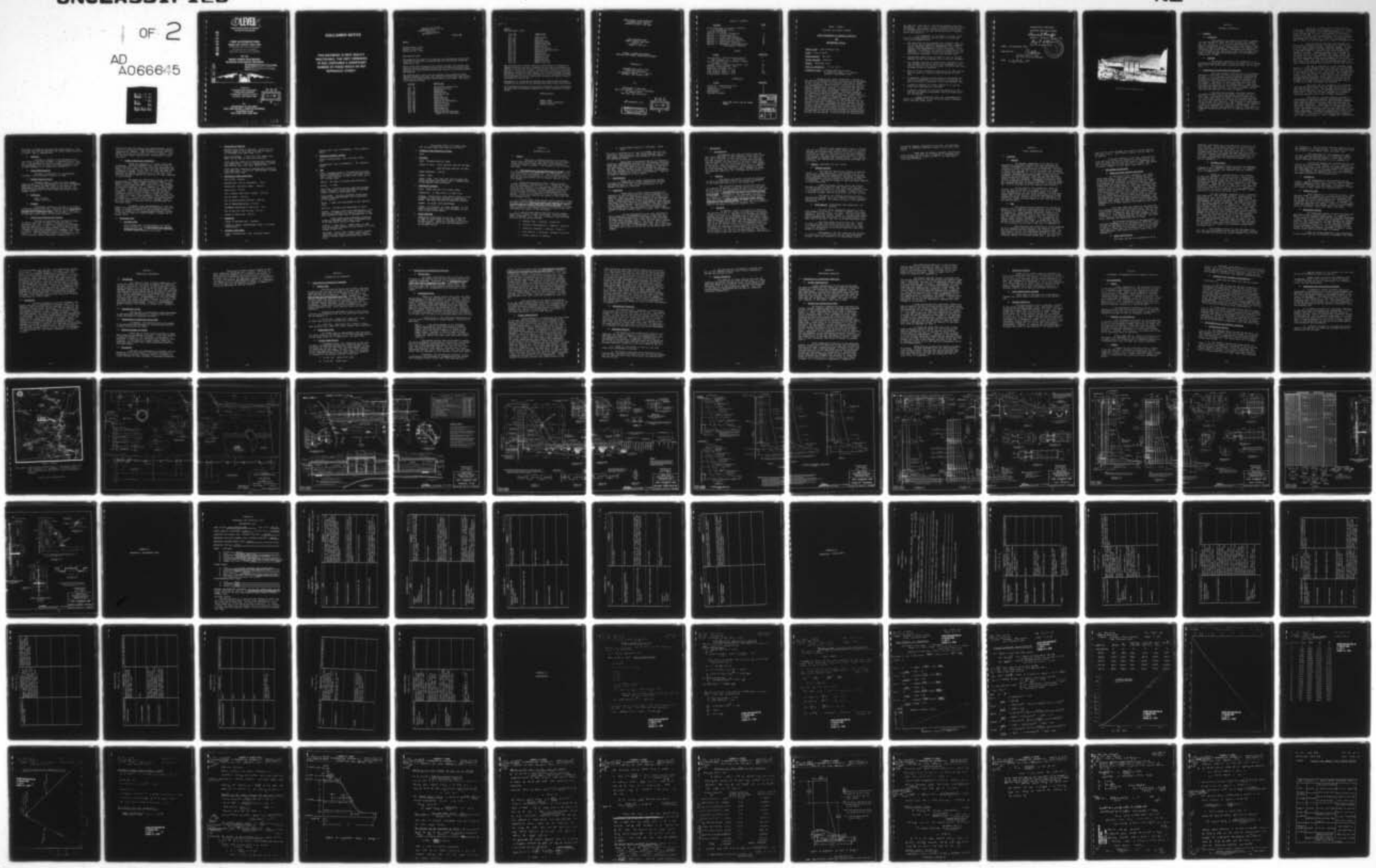

5 LEVEL II

UPPER HUDSON RIVER WATERSHED
SACANDAGA RIVER BASIN

LAKE ALGONQUIN DAM

HAMILTON COUNTY, NEW YORK

CONTRACT NO. DACW 51-78-C-0035

NY 172

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Prepared by
CONVERSE WARD DAVIS DIXON
CONSULTING ENGINEERS
91 ROSELAND AVENUE, P.O. BOX 91
CALDWELL, NEW JERSEY 07006

For

DEPARTMENT OF THE ARMY
NEW YORK DISTRICT, CORPS OF ENGINEERS
26 FEDERAL PLAZA
NEW YORK, NEW YORK 10007

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DEPARTMENT OF THE ARMY
U. S. ARMY ENGINEER DISTRICT, NEW YORK
26 FEDERAL PLAZA
NEW YORK, NEW YORK 10007

2 OCT 1978

NANEN-F

Honorable Hugh L. Carey
Governor of New York
Albany, New York 12224

Dear Governor Carey:

The purpose of this letter is to inform you of a clarification of the guidelines used by this office in assessing dams under the National Program of Inspection of Dams.

Office of the Chief of Engineers has recently provided a clarification that dams with seriously inadequate spillways are to be assessed as unsafe, non-emergency, until more detailed studies prove otherwise or corrective measures are completed.

The following dams in your state have previously been assessed as having seriously inadequate spillways, with capability to pass safely only the percentage of the probable maximum flood as noted in each report. They are now to be assessed as unsafe:

I.D. NO.	NAME OF DAM
N.Y. 59	Lower Warwick Reservoir Dam
N.Y. 4	Salisbury Mills Dam
N.Y. 45	Amawalk Dam
N.Y. 418	Jamesville Dam
N.Y. 685	Colliersville Dam
N.Y. 6	Delta Dam
N.Y. 421	Oneida City Dam
N.Y. 39	Croton Falls Dam
N.Y. 509	Chadwick Dam (Plattenkill)
N.Y. 66	Boyds Corner Dam
N.Y. 397	Cranberry Lake Dam
N.Y. 708	Seneca Falls Dam
N.Y. 332	Lake Sebago Dam
N.Y. 338	Indian Brook Dam
N.Y. 33	Lower(S) Wiccopee Dam (Lower Hudson W.S. for Peekskill)

NANEN-F
Honorable Hugh L. Carey

I.D. NO.	NAME OF DAM
N.Y. 49	Pocantico Dam
N.Y. 445	Attica Dam
N.Y. 658	Cork Center Dam
N.Y. 153	Jackson Creek Dam
N.Y. 172	Lake Algonquin Dam
N.Y. 318	Sixth Lake Dam
N.Y. 13	Butlet Storage Dam
N.Y. 90	Putnam Lake (Bog Brook Dam)
N.Y. 166	Pecks Lake Dam
N.Y. 674	Bradford Dam
N.Y. 75	Sturgeon Pool Dam
N.Y. 414	Skaneateles Dam
N.Y. 155	Indian Lake Dam
N.Y. 472	Newton Falls Dam
N.Y. 362	Buckhorn Lake Dam

The classification of "unsafe" applied to a dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean, however, that based on an initial screening, and preliminary computations, there appears to be a serious deficiency in spillway capacity so that if a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream from the dam.

Consequently, it is advisable to implement the recommendations previously furnished in the reports for the above-mentioned dams as soon as practicable.

It is requested that owners of these dams be furnished a copy of this letter and that copies be permanently appended to all reports previously furnished to you.

Sincerely yours,

CLARK H. BENN
Colonel, Corps of Engineers
District Engineer

UPPER HUDSON RIVER WATERSHED
SACANDAGA RIVER BASIN
HAMILTON COUNTY, NEW YORK

LAKE ALGONQUIN DAM
TOWN OF WELLS, NEW YORK
NDS # 172
NYSDEC # 171-2700

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY INSPECTION PROGRAM

Prepared by

CONVERSE WARD DAVIS DIXON
Consulting Engineers
91 Roseland Avenue, P. O. Box 91
Caldwell, New Jersey 07006

For

DEPARTMENT OF THE ARMY
New York District, Corps of Engineers
26 Federal Plaza
New York, New York 10007

27 September 1978

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PLATES

APPENDICES

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PHASE I REPORT

NATIONAL DAM SAFETY PROGRAM

BRIEF ASSESSMENT OF GENERAL CONDITION

AND

RECOMMENDED ACTION

Name of Dam: Lake Algonquin Dam

Owner: Town of Wells

State Located: New York

County Located: Hamilton

Stream: Sacandaga River

Date of Inspection: 19 July 1978

Inspection Team: Converse Ward Davis Dixon
91 Roseland Avenue, P. O. Box 91
Caldwell, New Jersey 07006

Based on our visual inspection, a review of the available engineering data, and calculations performed as part of this study, the Algonquin Lake Dam is judged to be in generally good structural condition and functioning satisfactorily at this time. Our hydrologic and hydraulic computations, however, indicate that the overflow spillway cannot pass the Probable Maximum Flood (PMF) without the dam being overtopped. Therefore, based on the screening guidelines established by the Department of Army, Office of the Chief of Engineers (OCE), the spillway capacity is rated as inadequate. In addition, the spillway is considered seriously inadequate since all the conditions established by the OCE guidelines for determining seriously inadequate spillway capacity are satisfied. Since this assessment was based on OCE screening criteria and approximate computational techniques, a detailed hydrologic and hydraulic evaluation of the watershed and gravity/spillway-gate/sluiceway structure should be performed by the use of more precise and sophisticated methods and procedures. Following such an investigation,

the need for, and type of, mitigating measures should be determined. Until such a study is completed and the spillway adequacy established, around-the-clock surveillance of the dam should be provided during periods of unusually heavy precipitation.

Our assessment of the general physical condition of the Lake Algonquin Dam has led us to make the following recommendations:

1. The efficiency of the upstream clay blanket, heel cut-off walls, and foundation drainage blanket to reduce uplift pressures should be determined. This would require field measurement of uplift pressure, by piezometers, for example, for specific elevations of the headwater. This study should be performed as soon as practicable, preferably within one calendar year.

2. Appropriate steps should be taken to stop or control seepage through the earthen fill at the left abutment.

3. All cracked, spalled and deteriorated concrete on the left abutment retaining walls, and elsewhere on the gravity/spillway and gate/sluiceway structure and right abutment should be repaired.

4. Repairs to the inoperative gate should be completed as soon as possible, certainly before the end of this year.

5. An emergency warning system should be formulated and officially presented to local police authorities as soon as possible, preferably within one calendar year.

6. A specific program for normal operation of the dam should be formulated and followed.

7. A specific program for periodic maintenance of the dam and its operating equipment should be established and implemented.

Unless otherwise noted, all recommendations should be implemented as soon as practicable, preferably within the next three years.

Respectfully submitted,

CONVERSE WARD DAVIS DIXON

Edward A. Nowatzki, Ph.D., P.E.

Gary S. Salzman, P.E.

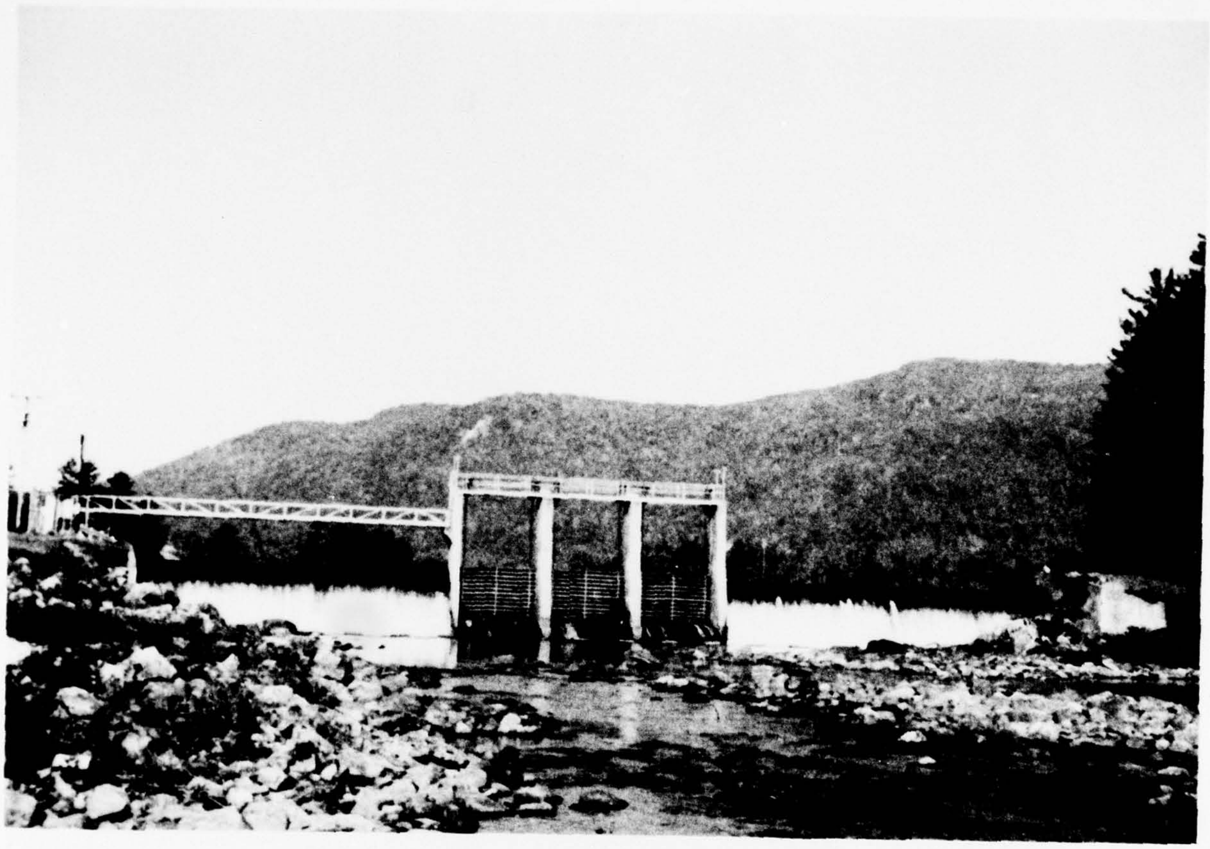
PROFESSIONAL ENGINEER
GARY S. SALZMAN
No. 40046
UNIVERSITY STATE OF NEW YORK

Date: 19 September 1978

Approved by:

Colonel Clark H. Benn
New York District Engineer

Date: 27 September 1978



OVERVIEW-LAKE ALGONQUIN DAM

SECTION 1

PROJECT INFORMATION

1.1 General

a. Authority

The authority to conduct this Phase I inspection and evaluation comes from the National Dam Inspection Act (P.L. 92-367) of 1972 in which the Secretary of the Army was authorized to initiate, through the Corps of Engineers, a program of safety inspections of non-federal dams throughout the United States. Management and execution of the program within the State of New York has been undertaken by the New York State Department of Environmental Conservation (NYSDEC).

b. Purpose

The primary purpose of the inspection is to evaluate available data and to give an opinion as to whether the subject dam constitutes a hazard to human life or property.

1.2 Description of Dam and Appurtenances

The Lake Algonquin Dam was built in 1958-1959, replacing a dam built in 1924 and partially reconstructed in 1949. It is a concrete gravity/spillway structure approximately 239 feet long from abutment to abutment, including a 66-foot long gate and sluiceway structure near its center. The dam is 18 feet high from spillway crest to bottom of base slab; it is approximately 17 feet high from spillway crest to the top of the clay blanket upstream. The right spillway section is 88 feet in length, and the left spillway section is 85 feet in length.

The gate and sluiceway structure consists of three vertical lift roller gates, 12 feet high by 19 feet long; a reinforced concrete sluiceway whose crest is 11 feet lower than the crest of the spillway sections; and four piers spanned by an operating platform that contains the gate lift controls. The platform is approximately 19 feet above the crest of the dam. Access to the operating platform is obtained only from the right abutment via a steel truss walkway that extends to pier #1 (the piers are numbered consecutively from 1 to 4, starting from the right pier).

There is a 1½-foot thick, 9-foot wide concrete apron downstream, that extends for the entire length of the dam. The apron is followed downstream by a 3½-foot wide concrete end sill that also extends for the entire length of the dam. The sill supports 3-foot by 4-foot by 1½-foot high triangular baffles spaced on 10-foot centers. Downstream from the end sill, there are approximately 30 feet of heavy rip rap followed by 20 feet of boulder paving.

The 1958 design drawings indicate that the reinforced concrete base slab of the dam is underlain by a 6-inch thick "select gravel" drainage blanket. In addition, at approximately 13½ feet from the upstream face of the dam, there is a gravel filter drain running the length of the dam. Pressure release in this system is provided through 2-inch diameter pipe weeps located over the drain at 6-foot centers along the length of the dam. A similar drainage system exists beneath the apron and end sill, except that the drainage blanket is 1 foot thick. The filter drain and weeps are located approximately 12½ feet farther downstream from those of the main dam section.

The dam is anchored by 1-inch diameter steel dowels on 1-foot centers along its length at the upstream end, to a concrete cutoff wall that had been built as part of the original dam in 1924. The cutoff wall extends about 5 feet below the bottom of the base slab, and is approximately 1-3/4 feet thick. At the time of the construction of the present structure, steel sheet piling was driven adjacent to and upstream of the existing cutoff wall to a depth of 25 feet below the bottom of the base slab or to the top of rock, whichever was shallower.

The right and left abutments are reinforced concrete retaining walls that had been built as part of the original dam in 1924. The present structure is apparently not structurally keyed into these walls (Refer to Plate VIII). The joint at each abutment is a standard cork-type expansion joint with a rubber waterstop.

Each gravity spillway section consists of two monolithic concrete sections, approximately equal in size, keyed to each other and to the reinforced concrete base slab horizontally; adjacent sections are keyed to each other vertically. There are two longitudinal construction joints with keys in each of the monolithic sections, one between the two parts of each monolith, and one at the base. The base slab sections are keyed to each other transversely (direction perpendicular to the axis of the dam). The apron and sill sections are keyed to each other longitudinally, and to themselves, transversely.

The apron is keyed to the base slab longitudinally. The right and left gravity sections are doubly keyed vertically to piers 1 and 4, respectively.

b. Location

The dam is located on the Sacandaga River just south of the Town of Wells, in Hamilton County, New York. The location of the dam is shown on Plate I, which was reproduced from the USGS 15-minute Quadrangle Sheet of Lake Pleasant, N.Y., N43°15'00", W74°15'00".

c. Size Classification

The dam is classified as "intermediate" (storage = 1200 acre-feet; height = 17 feet).

d. Hazard Classification

Because there is a New York State summer campsite about 2 miles downstream of the dam, and because there are a number of homes close to the banks of the Sacandaga River between the campsite and the dam, the hazard classification for the subject structure is considered "high".

e. Ownership

Town of Wells
Wells, New York

f. Purpose

The primary purpose of the dam is to create a lake for recreational use. According to the Application for the Construction of a Dam, filed in 1958 (Refer to Appendix E), a secondary purpose of the dam is to provide an auxiliary water supply for the Town of Wells.

g. Design and Construction History

The dam was designed for the Town of Wells in 1958 by the firm of Erdman, Anthony and Hosley, Rochester, New York, to replace a wooden crib structure that had been built in 1924. The original dam had been repaired and partially reconstructed in 1949 following a breach of the right abutment on 31 December 1948 (Refer to Morrell Vrooman Engineers Report of August 1949 - Appendix E). The 1958 design drawings and drawings by the firm of Morrell Vrooman Engineers, Gloversville, N.Y.,

relating to the 1949 repairs and reconstruction, are on file with the New York State Department of Environmental Conservation (NYSDEC). Some of these drawings are presented as Plates III through VIII in this report. There was no other formal design and/or construction history available.

h. Normal Operational Procedure

There are apparently no formal operational procedures. We were informed by Mr. John Orr, Town of Wells Supervisor, that the lake level was maintained adequately by the overflow spillway, and that the gates were infrequently opened, usually only to drain the lake for emergency or unusual situations (e.g., last year, the lake was drained to search for the body of a man suspected to have drowned).

The gates are electrically driven, and were manufactured by ARMCO Drainage and Metal Products, Inc., Denver, Colorado, model number LO42-3456C. The control panel for the right and center gates is located on shore at the right abutment (Fig. 1, Appendix D). The electrical supply to the gate platform is also controlled from this panel. The right and center gates are raised or lowered at a rate of approximately 6 inches per hour. A portable electric motor is used to raise or lower the left gate at a rate of approximately 5 feet per hour (Fig. 2, Appendix D). The left gate cannot be operated unless the other gate controls are shut off; however, there is an on-off switch on the gate control platform near the left gate stand.

It appears from correspondence on file with NYSDEC that operating procedures are haphazard, and could result in damage downstream if not regulated carefully (Refer to Appendix E for correspondence between citizens of the Town of Hope and NYSDEC between 8 January and 30 January 1978).

1.3 Pertinent Data

a. Drainage Area

The drainage area is approximately 261 square miles according to the Application for the Construction of a Dam, State of New York Department of Public Works, dated 31 July 1958.

b. Discharge at Damsite

Maximum known flood at damsite: 20,000 cfs (estimated based on flow data from gaging station on Sacandaga River near Hope, N.Y.).

Design discharge: 17,300 cfs (with gates fully open, and 3½ feet of water over spillway).

Total spillway capacity at maximum pool elevation with gates shut: 9,400 cfs (approximate; assumes gate section acts like sharp crested weir).

Total spillway capacity at maximum pool elevation with gates open: 24,000 cfs (approximate; assumes gates are fully open).

c. Elevations (feet above MSL)

Top of dam: 986.84.

Maximum pool (top of abutments): 992.0.

Normal pool (spillway crest): 986.84.

Gate sills: 975.84.

Top of gates (when fully closed): 987.84.

Top of piers: 1004.84.

Top of gate platform grating: 1005.97.

Top of downstream apron: 970.84.

Streambed downstream of end sill: 971±.

Top of end sill and rip rap: 971.34.

Bottom of base slab: 968.84.

d. Reservoir

Length of maximum pool: unknown.

Length of normal (recreational) pool: 1½ miles (approximate).

e. Storage (acre-feet)

Normal (recreational) pool (spillway crest): 1200.

Maximum pool (top of abutments): 2700 (approximate).

f. Reservoir Surface (acres)

Normal (recreational) pool (spillway crest): 275.

Maximum pool (top of abutments): 300 (approximate).

g. Dam

Type: Concrete gravity; two ogee-type spillway sections separated by a 66-foot long gate structure. Left spillway, 85 feet long; right spillway, 88 feet long.

Length: 239 feet (including gate structure).

Height: 17 feet.

Top width: Spillway section ogee-type rounded crest, nominally 6 feet from base of upstream slope at crest to downstream face.

Side slopes: Vertical upstream; curved downstream, slope approximately 5-7/8 horizontal to 12 vertical.

Apron: 9 feet wide downstream of main dam section.

End sill: 3½ feet wide downstream of apron.

Cutoff: Concrete cutoff wall approximately 1-3/4 feet thick extending to a depth of about 5 feet below bottom of base slab at upstream end of dam.

Steel sheet piling adjacent to and upstream of concrete cutoff wall to maximum depth of 25 feet below bottom of base slab.

Baffles: 4 feet long x 3 feet wide x 1½ feet high triangular concrete baffles spaced at 10-foot centers on end sill.

Drainage: 6-inch thick "select gravel" blanket under main dam section; 1-foot thick "select gravel" blanket under downstream apron and end sill.

Two filter drains with weep pipes, one row each under main section and apron.

h. Diversion and Regulating Tunnel

None.

i. Spillway

Type: Concrete gravity, ogee.

Length of weir: right spillway section, 88 feet.

left spillway section, 85 feet.

Crest elevation: 986.84.

Gates: None.

Piers: None. The right and left spillway sections are keyed into piers 1 and 4, respectively, of the central gate structure.

j. Regulating Outlets

Type: Three vertical lift roller gates.

Dimensions: 12 feet high by 19 feet long.

Closure: Electrically from control panel at right abutment or from gate platform on piers above gates. The gates can also be operated manually.

Access: To gates, from downstream.

Access: To controls, at right abutment, or via steel truss walkway from right abutment to platform on piers above gates.

k. Other Features

Immediately downstream of end sill, there are 30 feet of large stone rip rap and 20 feet of boulder paving. There is a natural rock outcrop in the center of the River just downstream of the boulder paving.

SECTION 2

ENGINEERING DATA

2.1 Design

There was a moderate amount of structural design data available for the subject dam and its appurtenant structures; there were very little hydraulic/hydrologic data available. The sources of the available data are:

a. Application for the Construction of a Dam, filed with State of New York Department of Public Works, on 31 July 1958 (Refer to Appendix E). This document, the 1924 application for construction of the original structure, and the 1949 application for its repair, are all on file with NYSDEC.

b. Three drawings dated August, 1949 by Morrell Vrooman Engineers, Gloversville, N.Y., regarding the repair and reconstruction of the original structure. These drawings also relate to the present structure since the right and left abutments, and the concrete cutoff wall of the original structure, were incorporated into its design. Plate II is a portion of one of these drawings, showing in plan the location and extent of the right and left abutment walls.

c. A brief report dated August 1949 by Morrell Vrooman Engineers entitled Report on Repair and Remodelling of Dam No. 544, Town of Wells, Hamilton County, New York. This report contains a general history of the original structure, some information and comments concerning flood flows, and a proposal for repair of the structure.

d. A set of eight design drawings for the present structure by Erdman, Anthony and Hosley, Consulting Engineers, Rochester, N.Y. The set is dated 19 June 1958 and contains the following drawings:

1) General Plan (1 drawing - Plate III)

2) Spillway Cross-Section (1 drawing - Plate IV)

3) Stability Diagrams (1 drawing - Plate V)

4) Pier Details (2 drawings - Plates VI and VII)

5) Walkway Details (1 drawing)

6) Miscellaneous Details (2 drawings - Plate VIII)

This set of drawings is on file with NYSDEC and with the Supervisor, Town of Wells. Only those drawings that contain useful information for the purpose of this report are reproduced herein.

There were no structural design or hydraulic/hydrologic computations available for the present structure. There were some computations for the design of the counterfort retaining walls along the right abutment of the original dam. Since a portion of that structure was used as the right abutment of the present structure, those computations were checked and found to be satisfactory. They are included in Appendix E of this report.

2.2 Construction

There were no formal construction records available for either the original construction in 1924, the repairs done in 1947, or the construction of the present structure in 1958/1959.

2.3 Operation

There were no formal records available of operation of the subject dam or of flow discharges at the damsite. There is a USGS gaging station near Hope, N.Y., about 3½ miles downstream of the damsite on the Sacandaga River. Records dating at least as far back as 1913 are available for that location, but measurements there include flows from both the east (Lake Algonquin) and west branches of the Sacandaga River. The west branch confluence with the Sacandaga River lies downstream of the subject dam. (Refer to Morrell Vrooman Report of August, 1949 - Appendix E.)

In the recollection of Mr. Orr, the Town of Wells Supervisor, the maximum height of water over the spillway since construction of the dam in 1958 was about 2½ feet above spillway crest (about half the way up the left and right abutment walls). He did not recall whether or not the gates were opened at that time. This would correspond to a flow of approximately 3200 cfs if the gates were shut, and a flow of 15,400 cfs if the gates were fully opened.

2.4 <u>Evaluation</u>

a. <u>Availability</u>

Engineering data were provided by the New York State Department of Environmental Conservation (NYS-DEC) and by the Town of Wells Supervisor, Mr. John Orr. The data provided by Mr. Orr had already been obtained from the NYSDEC, and consisted of the eight design drawings of 1958. Mr. Orr did arrange to have a gate tender at the site to open and close the gates (Fig. 2, Appendix D) on the day of the inspection, and attempted to have the Town Engineer, Mr. Paul Clairmont, present to answer any technical questions; Mr. Clairmont was not able to come.

b. <u>Adequacy</u>

The nature and amount of available engineering data are adequate to make a satisfactory assessment of the structural stability of the subject dam.

The available hydraulic/hydrologic data are not adequate to perform a detailed analysis of the dam's ability to pass the recommended Spillway Design Flood (SDF) as contained in <u>Recommended Guidelines for Safety Inspection of Dams</u>, Department of the Army, Office of the Chief of Engineers. Consequently, the assessment presented in this report is founded on approximate solutions based on data contained in <u>Upper Hudson and Mohawk River Basins Hydrologic Flood Routing Models</u> (October 1976), a report prepared for the Department of the Army, New York District, Corps of Engineers, by Resource Analysis, Inc.

c. <u>Validity</u>

In general, there is no reason to question the validity of any of the data obtained from the sources listed in Section 2.1. Because of an apparent discrepancy between the size of the drainage area as reported on the 1958 application for construction and in the Corps of Engineers hydrologic model study, the drainage area was identified on a composite of USGS quads and determined by planimeter to be 224 square miles. This corresponds closely to the value (261 sq. mi.) given in the application for construction and represents approximately 60% of the area of Subbasin 46 (377 sq. mi.) of the Upper Hudson River Basin given in the Corps of Engineers report. The value of 261 square miles was used in the hydrologic computations.

The only other questionable items of information are the value of the sliding coefficient of friction between the concrete base slab and the gravel underdrain contained on Plate V. The value as given seems unrealistically high, and could lead to higher factors of safety with regard to sliding than probably exist. This was considered in the computations and evaluation performed for this study.

2.5 Geology (performed for this study)

a. General Geology

The lake and damsite are located in Hamilton County, N.Y. The damsite is in the general vicinity of the contact between the Beekmantown and Saratoga Springs Group, Theresa and Hoyt formations (dolomites, limestones and sandstones); and the Trenton and Black River Group (limestones). The bedrock ranges in age from upper Cambrian to lower Ordovician.

There are normal faults within one mile of the dam on both the east and west sides, with the lake in the downthrown position. With this fault pattern, there is a potential for fault block movement. There is also a reported linement running north-south very near the dam.

The region has suffered glaciation during the Wisconsin stage, and a thin veneer of glacial deposits mantles the bedrock. The region is part of the glaciated Adirondacks.

b. Site Geology (Interpreted from stereo-pair air photos)

The soil cover immediately adjacent to the lake appears to be thick (> 10 feet); however, rock rises sharply to the east, south, and west. The rocks are metasediments, possibly folded. There is a rather large normal fault traceable the full length of the photos about 9,500 feet east of the dam. The lake is on the downthrown side.

The lake slopes are quite flat for a short distance and then rise rapidly due to high rock. There is a delta being built up on the west shore of the lake inlet with indications of siltation about 800 feet downstream of the delta.

Downstream of the dam, there may be boulders, or siltation, at the fold axis of the first downstream meander; there are boulders on the downstream slope of

the second meander downstream of the dam. The upstream channel is currently forming an oxbow in the wider flood plain.

There were no geologic features (stratification, faults, cavities, etc.) detected or suspected that could be expected to affect the dam or its appurtenant structures adversely.

SECTION 3

VISUAL OBSERVATIONS

3.1 Findings

a. General

The general appearance of the dam and its appurtenant structures suggests that the structure was formally engineered and that it has been maintained satisfactorily since its construction in 1958/1959. The Town of Wells built the dam out of its own funds, and currently maintains and operates it. In an interview with Mr. Nelson, a seasonal resident whose mobile home is near the left abutment, we discovered that there may be some question about ownership of property which provides the most ready access to the left abutment. Subsequent discussions with Mr. Orr revealed that he had no knowledge of such an ownership problem and that he doubted that such a problem existed. This is a matter which may require clarification.

At the start of the inspection, water was flowing over the spillway at a depth of 3½ to 4 inches. With the opening of the two operable gates, the water level dropped below spillway crest elevation, so the downstream face of the spillway could be inspected and joints checked for leaks. Before the end of the inspection, the gates were closed and water began to flow over the spillway section again.

b. Dam

The gravity/spillway sections of the dam appeared to be in generally good condition, with only minor erosion on the crest and downstream faces (Figs. 3, 4 and 5, Appendix D). Some moderate structural cracking was noticed on the downstream face of the right monolith of the left gravity/spillway section near its junction with pier 4 (Figs. 4 and 5, Appendix D). No leakage was observed coming through the cracks. A large spall was observed on the crest of the right gravity/spillway section at its junction with the pier 1 key (Fig. 6, Appendix D).

The monolithic and construction joints appeared to be in generally good condition; the monolith joint of the left gravity/spillway seemed to be slightly open. The para-plastic caps on the cork-sealed expansion joints between the gravity/spillway sections and the left and right abutments appear to have been eroded away. The same is

true for the joints between the gravity/spillway sections and piers 1 and 4. The joints, however, did not seem to be leaking.

With the lowering of the lake, flow over the right spillway section was observed to cease slightly before flow over the left spillway section, suggesting that the two sections may not be vertically aligned. The amount of offset, however, is very small (less than ¼ inch).

c. Appurtenant Structures

1) Gates and Gate Control Structure

At the start of the inspection, all three gates were shut. A slight amount of leakage was noted coming from beneath each gate. (See Overview Photo.) On the day of the inspection, the right gate was inoperable because the lift motor was down for repairs. The middle gate was raised about 1½ inches in 15 minutes from a switch in the control panel at the right abutment area (Refer to Section 1.2h and Fig. 1, Appendix D). The left gate was raised a total of about 4 feet in approximately 45 minutes by an operator with a portable electric motor (Fig. 2, Appendix D). This was done in order to lower the lake level below spillway crest elevation so that the downstream face of the spillway could be inspected. The left and middle gates and gate controls functioned satisfactorily and, except for some rust on the lift arms, they appeared to be in good condition and generally well maintained.

The steel frame and grates of the gate control platform atop the gate structure piers, and the steel truss access walkway over the right spillway section, appeared to be in generally good condition (Figs. 2 and 20, Appendix D). The walkway from the left abutment to pier 4, as shown on the design drawings (Plate III), was never built, although the foundations are in place.

The concrete portions of the gate control structure appeared to be in satisfactory condition. Minor erosion was observed on the sluiceway and up to a height of about 3 feet above the sluiceway on each of the piers. There was a large spall on the right wall of pier 4 and some moderately sized spalls on piers 3 and 4 (Fig. 4, Appendix D).

2) Apron and End Sill

The apron and end sill appeared to be in

generally good condition, with some minor erosion of surface concrete. Drain weeps were clearly visible and no water was noticed flowing from them. In fact, a small vortex was noticed going into one of the drains of the concrete apron after the gates had been shut and the tailwater had dropped, and before flow had resumed over the spillway. The concrete baffles were very effective in dissipating energy when the gates were opened (Figs. 7 and 21, Appendix D).

3) Abutment Walls

The upstream wingwall of the right abutment appeared to be in generally good condition. A makeshift staff gage was located just upstream from the wingwall (Fig. 8, Appendix D).

There was a patched crack in the sidewall of the right abutment, near the spillway, running from the base of the wall up to its top (Fig. 9, Appendix D). A section of the wall near the junction of the spillway and apron was badly spalled (Fig. 9, Appendix D) and steel was exposed. There was also a weep hole in the wall that apparently had been flowing for some time since there was a definite water stain on the wall (Fig. 9, Appendix D). The concrete of the wall pedestal was moderately eroded, especially at and below the water line.

The left abutment appeared to be in generally poor condition. The upstream wingwall (perpendicular to direction of flow) had a vertical crack that extended transversely over the top of the wall. There was a slight displacement of the wall upstream. The edge of the wingwall was badly cracked and spalled, and a reinforcing bar was clearly exposed (Fig. 10, Appendix D). The left abutment sidewall (parallel to direction of flow) was moderately eroded at and below the spillway water line. There were large areas that had apparently been patched recently. Seepage was occurring from a large spall about 5 feet downstream of the gravity/spillway joint and approximately 2/3 of the way from the top (Figs. 11 and 12, Appendix D). There was also a large spall at the junction of the sidewall and downstream wingwall near the base (Figs. 11 and 13, Appendix D). Water was seeping through the spall, and reinforcing steel was clearly visible. The concrete at the base of the sidewall was moderately eroded below the water line.

The abutment section of the downstream wingwall was likewise observed to be leaking and badly spalled; the wire mesh reinforcing was clearly exposed (Figs. 14 and

15, Appendix D). The structural concrete beneath the wire mesh appeared to be badly eroded. Erosion was also noted at the base of the wingwall below the water line.

The retaining wall that extends for about 150 feet farther downstream of the downstream wingwall was seriously spalled and eroded; in some areas it had almost completely deteriorated (Fig. 16, Appendix D).

Significant seepage was observed flowing over the top of the downstream wingwall (Fig. 14, Appendix D), through the spalls in the sidewall and downstream wingwall, and from weep pipes built into the wingwall and retaining wall downstream of the left abutment (Fig. 17, Appendix D). This seepage was apparently coming through the earth embankment retained by the abutment walls.

d. Foundation

The foundation of this structure was not visible. Design drawings show a "select gravel" drainage blanket immediately below base slab. Soil boring data (Plate III) indicate that the blanket is probably founded on a layer of sand, gravel and inorganic silt.

e. Reservoir Area

Lake Algonquin is in the Town of Wells. There are many homes and commercial buildings along the eastern shoreline. The western shoreline is also developed, but not as heavily as the eastern shoreline. State Highway 30 bridges the lake at its northern end. In general, the slopes are shallower than about 1 vertical to 8 horizontal along the shoreline, but steepen quickly to about 1 vertical to 3 horizontal within 1000 feet of the lake.

f. Downstream Channel

The downstream channel is about 300 feet wide and contains many large boulders (Fig. 18, Appendix D). There is a rock formation approximately 100 feet downstream in the center of the Riverbed that is apparently a bedrock outcrop. The State Highway 30 bridge crosses the channel about 500 feet downstream of the dam, and does not appear to be a serious constriction. It was observed, however, that the upstream wingwall of the right abutment of this bridge was displaced laterally upstream by about 5 inches along a large vertical crack near its center.

There are three houses and a tent downstream of the bridge that may be affected by extremely high flows.

One of the houses, and the tent, were immediately adjacent to the River. A lumber company is located in the flood plain, but the premises appeared to be abandoned; this facility would probably be seriously damaged in the event of a flood. About two miles downstream, there is a public campsite and beach operated during the summer months by the State of New York. Due to the increased flow caused by opening the gates of the Lake Algonquin Dam on the day of the inspection, the water level at the beach rose about 1½ feet in less than an hour. Although no damage was done at the beach or to a small dam used for the temporary impoundment of water in the stream at the campsite (Fig. 19, Appendix D), the camp director was reportedly concerned that the structure might fail if the flow continued to increase. This campsite would be most seriously affected in the event of a major flood.

3.2 Evaluation

With the exception of the left abutment, the subject dam and its appurtenant structures seem to be in generally good condition and are expected to function satisfactorily under normal conditions. The concrete of the left abutment retaining walls (sidewalls and upstream and downstream wingwalls) is badly in need of repair. There are large spalls, and water is seeping through the concrete. In some areas, the previously-made repairs have themselves deteriorated. This suggests that the symptom and not the cause of the problem was treated in the past. There is apparently a considerable amount of seepage occurring through the earthen fill behind the retaining walls. This seepage, if allowed to persist, will continue to deteriorate the concrete of the retaining walls, and any efforts to provide only cosmetic repair will be futile; this seepage, if uncontrolled, may also result eventually in a piping failure.

SECTION 4

OPERATIONAL PROCEDURES

4.1 Procedures

Mr. John Orr, Town of Wells Supervisor, indicated that there are no formal procedures for operating the dam. Ordinarily, the level in Lake Algonquin is maintained naturally at or close to the spillway crest level. Emergency situations have arisen in the past that required the lake level to be lowered (refer to Section 1.2h); however, there is apparently no set procedure for this operation, and the Town Supervisor seems to be the responsible party in those cases. Correspondence on file with NYSDEC indicates that there have been occasions in the past when the gates were opened apparently without due regard to possible effects downstream. (Refer to correspondence dated 8 January 1978 through 30 January 1978 - Appendix E.)

4.2 Maintenance of Dam

The dam and its appurtenant structures appear to be maintained satisfactorily, although there was no formal maintenance procedure disclosed.

4.3 Maintenance of Operating Facilities

In general, the operating facilities appear to be maintained satisfactorily, although some rusting of gate lifter arms was noted.

4.4 Warning Systems in Effect

There are no formal warning systems or emergency operating procedures in effect. There are apparently back-up systems for the operation of the gates. As indicated in Section 1.2h, the gates are controlled electrically. In the event of a power outage, there are reportedly hand cranks and a gasoline-powered generator available nearby.

4.5 Evaluation

The dam, its appurtenant structures, and the operating facilities appear to have been satisfactorily maintained in the past, although there is currently no formal program for their regularly scheduled maintenance.

There are no formal warning systems or emergency operating procedures now in effect. The Town of Wells Supervisor seems to be in charge of and solely responsible for overseeing the entire project, from the physical condition of the dam itself to all the procedures for both its ordinary and emergency operation. This is considered to be an undesirable situation, as an emergency may arise when the one responsible person is absent; it should be rectified as soon as possible.

SECTION 5

HYDRAULICS AND HYDROLOGY

5.1 Evaluation of Hydraulic Features

a. Design Data

The spillway and gates were formally designed to pass 17,300 cfs with the gates fully open, and the estimated extreme high water at elevation 990.34 feet ±, 3½ feet over the top of spillway. (Refer to Plate III and Application for Construction of a Dam, Appendix E.) There was no information regarding peak flows at the damsite, although some flow data for the Sacandaga River downstream of the damsite are available in the August 1949 report of Morrell Vrooman Engineers (Refer to Appendix E). Unfortunately, those data include contributions of the West Branch Sacandaga River, which does not drain into Lake Algonquin.

Computations performed as part of this study indicate the following flows for the conditions noted (Refer to Appendix C):

1) 24,000 cfs. Gates fully open with 5 feet of water over spillway (maximum pool elevation).

2) 9,400 cfs. Gates shut with 5 feet of water over spillway and 4 feet over gates (maximum pool elevation).

b. Experience Data

No formal data or measurements were available for total flows at the damsite, or for the discharge ratings of the gates singly or in combination with each other.

c. Visual Observations

According to Mr. Orr, Supervisor of the Town of Wells, the maximum observed flow since the dam's construction in 1958/1959 occurred with approximately 2½ feet of water over the spillway. He could not recall whether or not the gates were opened at that time. The following flows, computed as part of this study (Refer to Appendix C), pertain to the maximum observed conditions:

1) 15,420 cfs. Gates fully open.

2) 3,180 cfs. Gates shut.

5.2 Evaluation of Hydrologic Features

a. Design Data

No formal hydrological data or analyses could be found in the records for the Lake Algonquin Dam and its immediate watershed. According to the Recommended Guidelines for Safety Inspection of Dams, Department of the Army, OCE, the recommended Spillway Design Flood (SDF) for the subject dam is the Probable Maximum Flood (PMF), since the dam is of intermediate size and poses a high hazard.

b. Experience Data

There is a gaging station near Hope, New York, approximately 3¼ miles downstream of the damsite. Records of measurements of maximum flows for that station since at least 1913 are available from the USGS. As indicated previously, however, these measurements are for a drainage area of 491 square miles and include flows from both the east and west branches of the Sacandaga River. (Refer to Morrell Vrooman Engineers report of August 1949, Appendix E.) Apparently, some unreported scaling factor was applied to the peak discharge of 32,000 cfs measured at the station on 27 March 1913 to arrive at the design flood of 17,300 cfs.

A description of the watershed characteristics is also given in the Morrell Vrooman report. The point is made that:

> "due to . . . the large pondage in the different lakes, the unusually large percentage of wooded area of sand and gravelly soil, the stream is not flashy nor large floods frequent in spite of the steep slopes. Because of the altitude and dense woods, over practically all of the area, the Spring floods are lighter and later than they would otherwise be."

The gaging station data would seem to support the above statement, since there is relatively little variation in maximum instantaneous peak flow or daily peak flow over a fourteen-year period from 1934 to 1948. However, the data would also seem to indicate that a long-recurrence-interval storm (greater than, say, the 100-year storm) was not measured at this station in that time period.

Therefore, the hydrological analysis in this investigation was performed by transposing to the subject basin Standard Project Flood (SPF) data obtained for a

larger, inclusive basin from the Upper Hudson and Mohawk River Basins Hydrologic Flood Routing Models, a report prepared for the New York District of the U.S. Army Corps of Engineers (USACE) by Resource Analysis, Inc. In that investigation, the rainfall-runoff mathematical model HEC-1 was used to reconstitute major historical floods in the basins under study, and to simulate the Standard Project Flood (SPF). In addition to the SPF simulation, the rainfall pattern for Tropical Storm Agnes was transposed to fall directly on the basins under study, and the discharges resulting from this rainfall were determined by an application of the model calibrated by comparison with available gage data. In a telephone conversation with Mr. Thomas Smyth, USACE New York District, we were informed that for Phase I hydrologic analyses, the Probable Maximum Flood (PMF) could be regarded as twice the SPF.

Data contained in the report for Subbasin 46 of the Upper Hudson River Basin from its source to its confluence with the Sacandaga River were interpolated to the Lake Algonquin watershed, which is contained within Subbasin 46. Flood routing computations in Appendix C indicate that the SDF is approximately 71,000 cfs. As indicated previously, this is also the PMF.

c. Visual Observations

Lake Algonquin is located near the confluence of the east and west branches of the Sacandaga River. Almost the entire watershed of the east branch of the Sacandaga River drains into the lake. From the hydrological computations, it does not seem that the lake is capable of significantly attenuating the flow of the Sacandaga River. (Refer to computations in Appendix C.) Our conversation with Mr. Orr appears to verify this assessment. He indicated that normally the lake could be drained in about 24 hours, if the three gates were left fully open; he also reported that it would take only a day or so to refill the lake under normal conditions. Our visual observations also support that assessment. At the start of the inspection, water was flowing at a depth of about 3½ to 4 inches over the spillway; all gates were shut and the end sill baffles were clearly visible (Fig. 20, Appendix D). The middle gate was then raised about 1½ inches and the left gate about 1½ feet. After approximately 40 minutes from the start of the gate openings, the water level over the spillway dropped to about 1 inch, and the tailwater rose about 2½ feet, completely submerging the baffles (Fig. 21, Appendix D). After another 40 minutes, during which time closure of the

gates had begun, the water ceased flowing over the spillway, and the tailwater rose to its maximum height of about 3-1/3 feet above its level at the start of the inspection. Within an hour of gate closure, the tailwater had dropped to an elevation below that at the start of the inspection, and water was beginning again to flow over the spillway. During this time, lakefront and downstream residents arrived at the damsite. The former indicated to us that they had noticed the lowering of the lake level when their small boats began to hang up on their docks; the latter informed us that there had been a rapid, substantial rise (> 1 foot) in the stream level at their campsites. It would appear, then, that should a large magnitude flow occur on the east branch of the Sacandaga River, upstream of Lake Algonquin, the water level in the lake would rise very rapidly, especially if the gates were not opened, and the flow pass through the lake virtually unattenuated. The computations in Appendix C verify this assessment of the visual observations (Refer to inflow-outflow hydrographs in Appendix C).

d. Overtopping Potential

The computations in Appendix C indicate that the subject dam will be overtopped by the PMF. The maximum height of water that can flow over the spillway section is 5 feet. At that height, the spillway passes 7640 cfs and the gate section 1760 cfs (assuming the gates to be shut), for a total of 9400 cfs. The routed PMF is 69,750 cfs. Therefore, the spillway can pass only 13.5 percent of the PMF. Even with all three gates fully open, the spillway and gate sections can pass only 24,000 cfs, or approximately 34 percent of the PMF without the dam being overtopped.

e. Spillway Adequacy

The results of the hydrological analysis indicate that the spillway capacity of the subject dam is inadequate with respect to passing the recommended SDF without overtopping the dam. In addition, the spillway is considered to be seriously inadequate because it satisfies all of the following conditions set forth in DAEN-CWE-HY Engineer Technical Letter No. 1110-2-234 dated 10 May 1978:

1) There is high hazard to loss of life from large flows downstream of the dam.

2) Dam failure resulting from overtopping would significantly increase the hazard to loss of life downstream from the dam from that which would exist just before overtopping failure.

3) The spillway is not capable of passing one-half of the Probable Maximum Flood without overtopping the dam and causing failure.

f. <u>Hazard Potential</u>

The quantity of water passing over the Lake Algonquin Dam during the PMF would be sufficient to create heavy and potentially dangerous flows on the banks of the Sacandaga River in the vicinity of the dam and downstream in the area of the State-operated campsite. The "high" hazard potential designated for the dam is, therefore, considered appropriate.

SECTION 6

STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observations

Visual observations of the gravity/spillway and gate structures did not reveal any signs of structural instability. The horizontal alignment appears to have been maintained. On the day of the inspection, flow over the right spillway section was observed to cease slightly ahead of the flow over the left spillway section when the lake level was lowered. This suggests that the two sections may not be vertically aligned. The amount of offset, however, is very small, less than ½ inch.

b. Design and Construction Data

No stability computations were available for review; however, summary stability diagrams were presented as part of the design drawings for two cases (Refer to Plate V). Case 1 considers the water level at elevation 986.84 (spillway crest); Case 2 assumes the water level to be at elevation 990.34 (3½ feet above spillway crest). Overturning and sliding stability of both the gravity/spillway section and gate/pier section were analyzed for each case. In the analysis of the gravity/spillway section under Case 1 conditions, an ice pressure of 4.5 k/ft at elevation 984 was assumed; in the overturning stability analysis, the resultant of forces was reportedly found to act through the middle third of the base at elevation 968.84. In the analysis of the gate/pier section under Case 1 conditions, an ice pressure of 4.0 k/ft at elevation 984 was assumed; in the overturning stability analysis, the resultant of forces was reportedly found to act through the middle third of the base at elevation 968.84. Therefore, in both instances, the factor of safety is greater than 1.

Since overturning stability of the gate/pier section is more critical than that of the gravity/spillway section under Case 1 conditions (the gate/pier section has less mass to offer resisting moment), a check on the overturning stability of the gate/pier section was performed as part of this study. In our analysis, a triangularly distributed uplift pressure equal to one half the hydrostatic pressure was assumed; the resultant of foces was found to lie at about the 1/3 point of the base (Refer to page 7 of 9 of stability computations, Appendix C).

The overturning stability of the spillway gravity section was analyzed for Case 2 (which is more critical than Case 1 for gravity section) and the location of the resultant was found to be well within the middle third of the base. (Refer to page 4 of 9 of stability computations, Appendix C.)

The design drawings (Plate V) indicate that the sliding stability of the gravity/spillway section under Case 2 conditions, and that of the gate/pier section under Case 1 conditions, are critical. Consequently, the factors of safety with respect to sliding were checked for both cases and were found to be 1.5 for the former (page 8 of 9 of stability computations, Appendix C) and 1.4 for the latter (page 9 of 9 of stability computations, Appendix C). The unrealistically high values of required friction coefficient as shown on Plate V were compensated for in our computations by considering the effect of passive soil resistance due to the upstream cutoff wall. Uplift pressure equal to one half of hydrostatic pressure was also assumed in our analysis.

Since the maximum flow conditions (water over dam at elevation 991.84) are more severe than those considered by Case 2, the factor of safety with respect to sliding was recomputed for the maximum flow condition, and found to be 1.3 (page 9 of 9 of stability computations, Appendix C). The same assumptions were made regarding passive resistance and uplift pressure as were made in our analyses of Cases 1 and 2 described above. Stability against overturning was not re-evaluated for maximum flow conditions because it was felt that an additional 1½ feet of water would not substantially change the location of the resultant force as computed for Case 2.

It should be noted once more that a hydrostatic pressure intensity factor of 1.0 for uplift was used for computation of line of pressure and sliding coefficients for the base of the dam, as shown in Plate V (the original computations). Because of the presence of a clay blanket upstream, sheetpiles and concrete cutoff walls at the heel of the dam, and a select gravel drainage blanket and filter drains beneath the base of the dam, an intensity factor of 0.5 was used for hydrostatic uplift in our computations.

There were no construction records available for review. We were informed by Mr. Orr that the gate tower access walkway over the left gravity/spillway section was never built, although the design drawings show that there should be one, and the foundations are in place (Fig. 11, Appendix D).

c. Operating Records

There are no formal operating records from which to evaluate the stability of the subject structure. The factors of safety with respect to overturning and sliding as originally computed for the design overflow of 3½ feet (and subsequently recomputed by us with an overflow of 5 feet) appear to be satisfactory within the assumptions made. These factors of safety would be larger for the maximum observed flood which reportedly occurred at a spillway overflow of 2½ feet as indicated previously.

d. Post Construction Changes

There were no reported post construction changes that would affect the stability of the subject dam.

e. Seismic Stability

The Lake Algonquin Dam is nominally located on the border between Seismic Zone 1 and Seismic Zone 2 according to the Algermissen Seismic Risk Map. The USACE guidelines suggest that in the event of doubt about the proper zone, the higher zone should be used. Although earthquakes that cause moderate damage can be expected to occur in Zone 2, the design and construction practices conventionally used for small concrete gravity dams are considered to be adequate in areas of low seismicity, and the safety factors used for static conditions should preclude major damage for all but the most catastrophic earthquakes. However, no computations were performed to evaluate the effect of earthquakes on the subject dam.

SECTION 7

ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Safety

Visual inspection of the system and a review of the available engineering data indicate that the dam is in generally good condition and functioning satisfactorily at this time. There is no evidence to indicate the existence of presently unsafe conditions, although computed factors of safety with respect to sliding are lower than those recommended in the OCE guidelines. There is an apparently bad seepage condition at the left abutment which, if left uncorrected, could lead to serious deterioration of the abutment's concrete retaining walls, and may eventually result in a piping failure.

Our approximate hydrologic/hydraulic calculations indicate that the discharge capacity of the dam, regardless of the position of the gates, is seriously inadequate according to the OCE screening criteria.

b. Adequacy of Information

The information available to us was adequate to perform fairly detailed analyses of the structural stability of the dam under assumed conditions of water overflow and uplift pressure. These data are sufficient, in conjunction with the results of the visual inspection, to make a reasonable assessment of the system's present condition. However, verification of the magnitude of the uplift pressures would be desirable.

Since there were no direct hydrological data available, our assessment of the overtopping potential is based solely on interpolation of modelling results for a drainage basin that includes the subject watershed.

c. Urgency

Inasmuch as the discharge capacity appears to be very seriously inadequate according to the OCE screening criteria, and since the downstream area contains homes and a popular public summer campsite, there is some urgency in performing the additional study recommended below.

Likewise, since deterioration of the concrete walls at the left abutment could lead to serious structural damage, and since this deterioration will continue as long as the seepage problem is not corrected, there is also some urgency in performing the repairs recommended below.

d. Necessity for Further Investigations

There are two areas that require further investigation:

1) In view of the very serious inadequacy of the dam to pass even one half of the PMF without the occurrence of overtopping, a detailed hydrologic and hydraulic evaluation of the watershed and the spillway/gravity and gate/sluiceway system should be performed using more precise and sophisticated hydrological/hydraulic methods and procedures. This further investigation should be performed as soon as possible. Following this study, the need for and type of mitigating measures should be determined. Until such a study is completed, around-the-clock surveillance of the structure should be provided during periods of unusually heavy precipitation.

2) Since stability computations are very sensitive to values of uplift pressure in the case of concrete gravity dams, and since certain assumptions regarding that pressure were made in the computations for this study, a field study should be performed to measure actual uplift pressures in the gravel drainage blanket under at least one headwater elevation. This study should be performed as soon as practicable, preferably within one year's time.

7.2 Recommendations and Remedial Measures

a. Alterations/Repairs

1) The seepage coming through the earthfill portion of the left abutment should be stopped or the embankment properly drained. Consideration should be given to the installation of a sheetpile or grout curtain cutoff wall, or, alternatively, the construction of a filtered subdrainage system.

2) All cracked, spalled and deteriorated concrete at the left abutment and elsewhere on the structure (including the right abutment) should be repaired with special attention being given to those areas where reinforcing steel is exposed. The extent of steel corrosion should be determined and more steel added where required.

3) Ongoing repairs to the inoperative right gate should be completed as soon as possible.

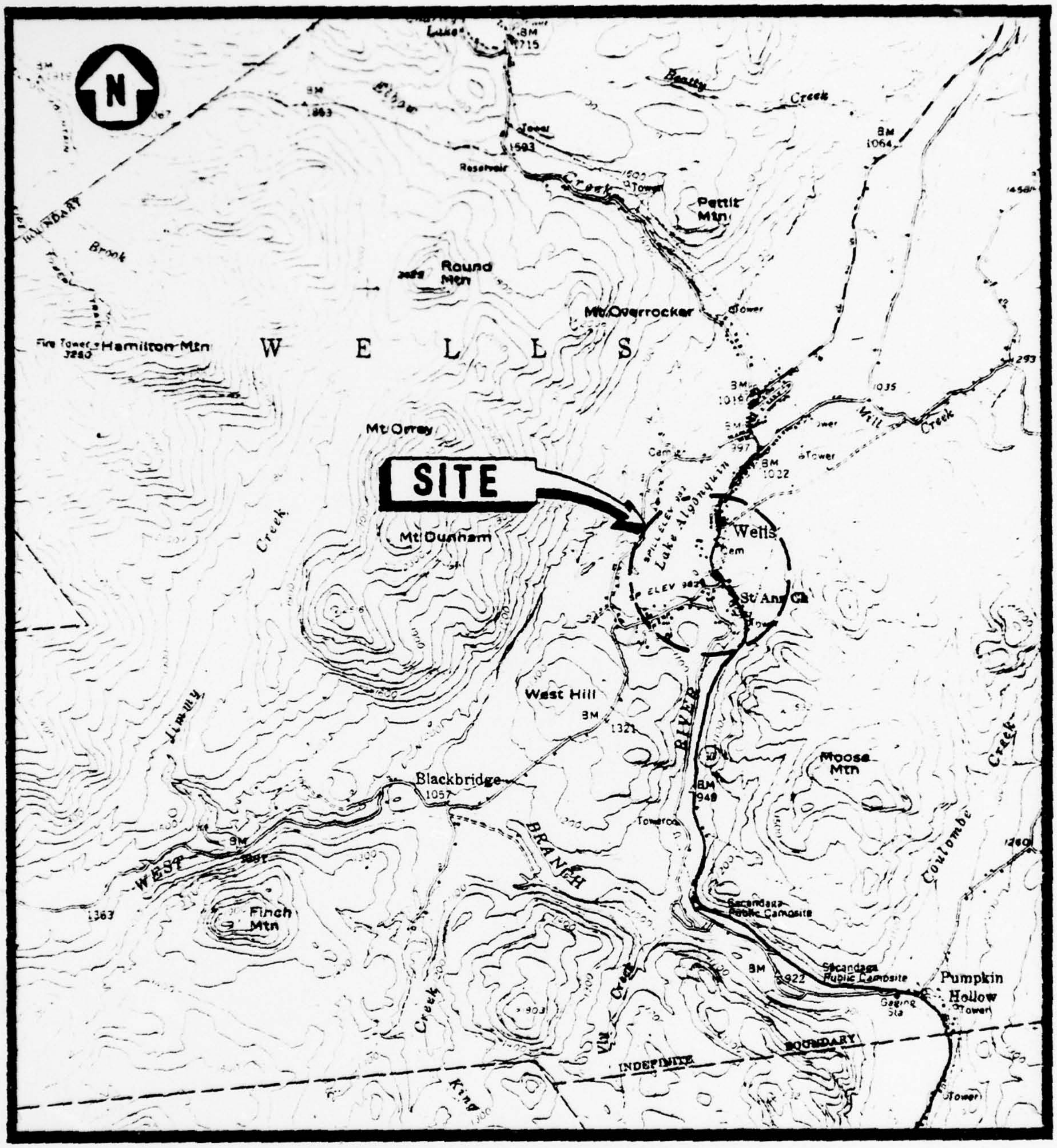
Except where indicated, the remedial work recommended above is not critical in terms of urgency. It should be done as soon as practicable, but certainly within the next three years.

b. Operations and Maintenance Programs

1) An emergency warning procedure should be formulated in coordination with local law enforcement and emergency rescue authorities. This document should contain chain-of-command names and telephone numbers in the case of an emergency. Consideration should be given to methods of implementation, in the event that telephone lines are down, roads closed, etc. The emergency warning procedure should be developed and officially presented to the authorities as soon as possible, preferably within one calendar year.

2) A specific program for the normal operation of the dam should be developed and implemented. In this program, the duties of responsible parties should be clearly defined. Specific operational procedures should be developed for various seasonal conditions, e.g. lowering of the lake level during the winter months. The effect of these procedures on the rights of lower riparian users of the Sacandaga River should be evaluated before such procedures are implemented.

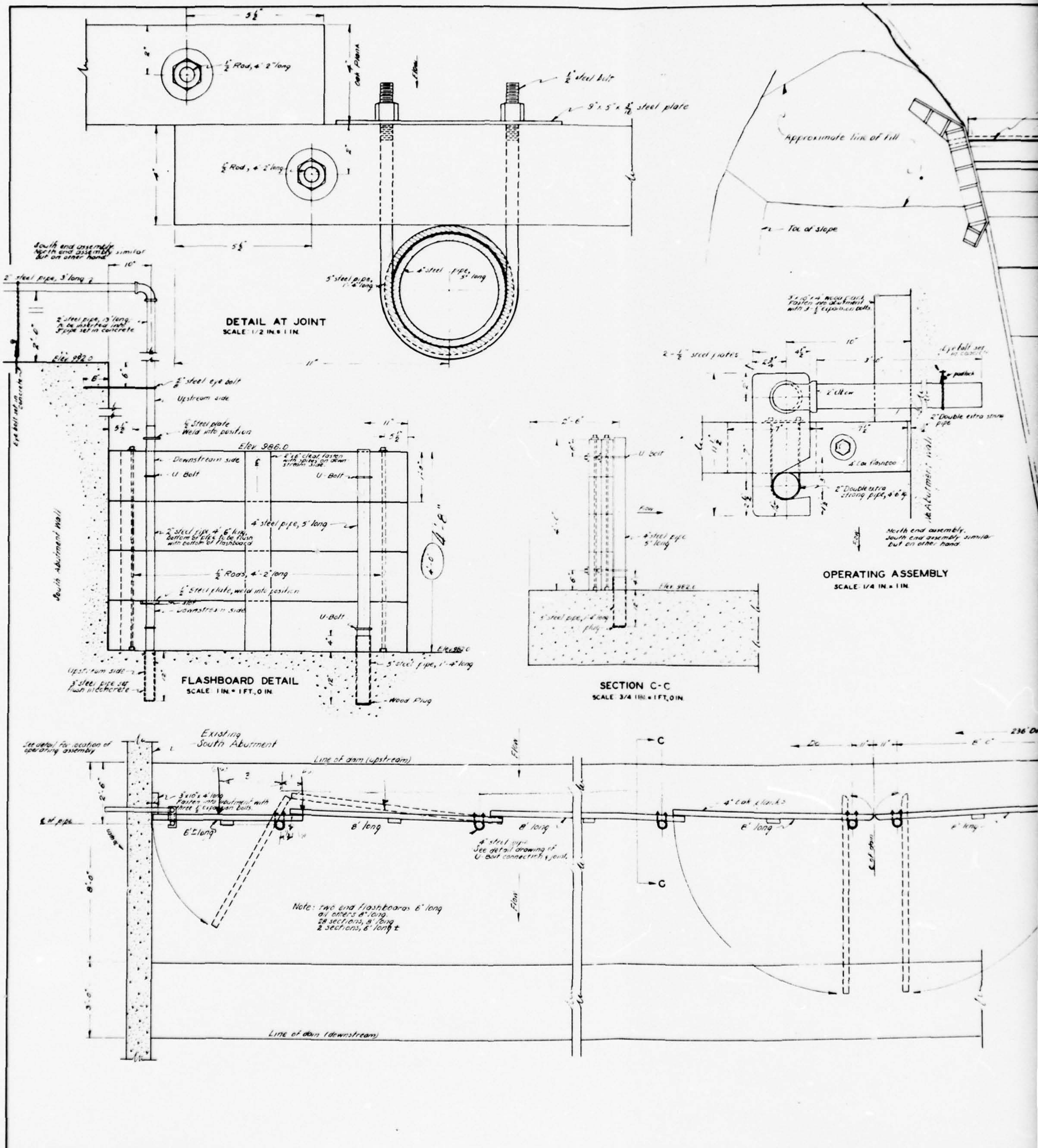
3) A specific program for the periodic maintenance of the dam and its operating equipment should be established and followed.



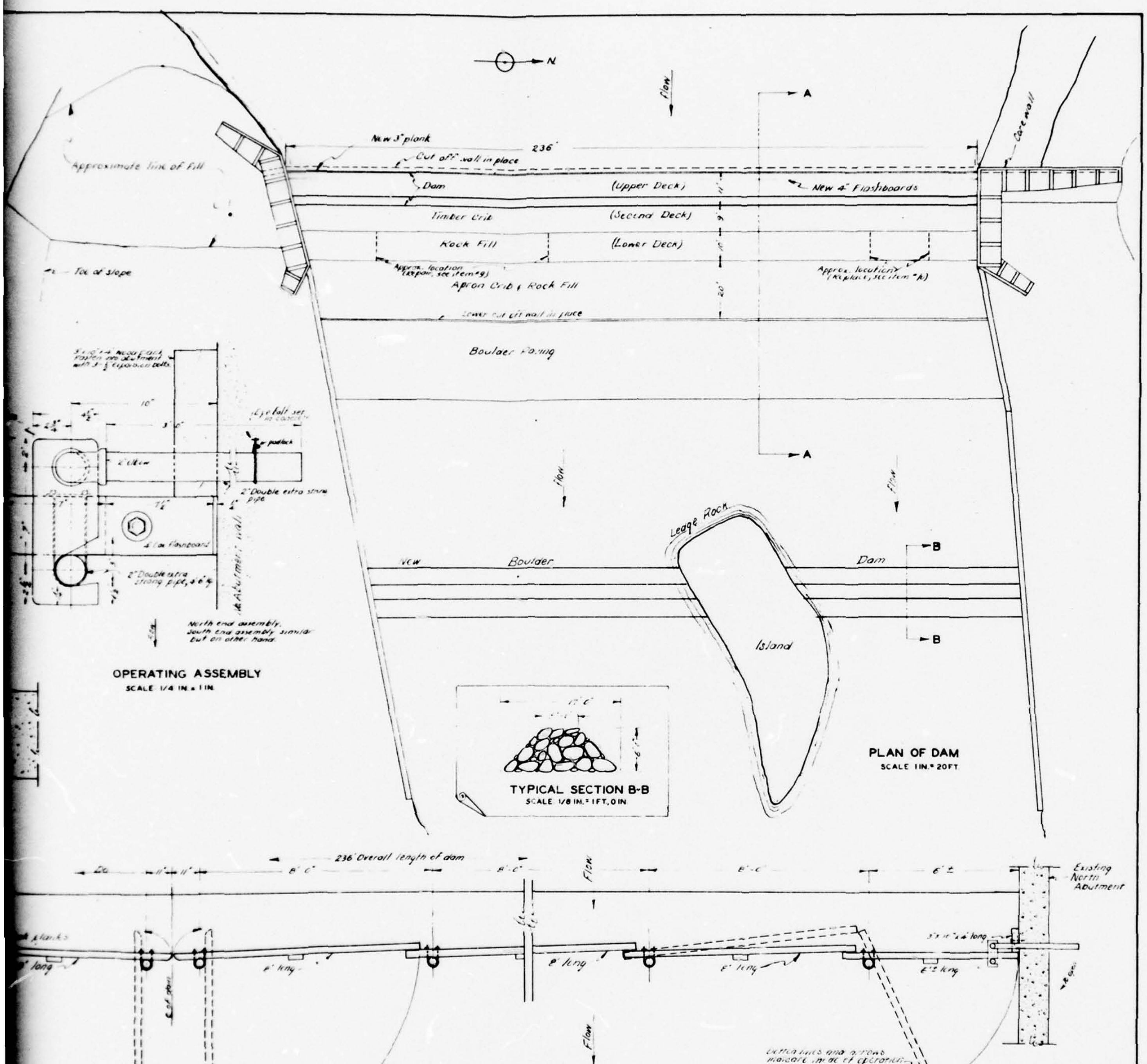
SCALE: 1"=4000'

MAP SOURCE: BASE MAP WAS ADAPTED FROM U.S. GEOLOGICAL SURVEY MAP, LAKE PLEASANT, N.Y. QUADRAGLE, 15 MINUTE SERIES, 1954. (BASE MAP MAY NOT REFLECT RECENT CARTOGRAPHIC CHANGES)

PLATE I-SITE LOCATION MAP



DETAIL AT JOINT
SCALE: 1/2 IN. = 1 IN.
½" Rod, 4'-2" long
½" steel bolt
9" x 5" x ¾" steel plate
5" steel pipe, 1'-6" long
4" steel pipe, 5' long
Approximate line of fill
Toe of slope
South end assembly. North end assembly similar but on other hand.
2" steel pipe, 3' long
2" steel pipe, 3' long, to be inserted into 3" pipe set in concrete
Elev. 992.0
⅝" steel eye bolt
Upstream side
½" Steel plate. Weld into position
Elev. 986.0
Downstream side
U-Bolt
2" steel pipe, 4'-6" long. Bottom of elbow to be flush with bottom of flashboard
4" steel pipe, 5' long
½" Rods, 4'-2" long
½" Steel plate, weld into position
South Abutment Wall
3" steel pipe set flush in concrete
5" steel pipe, 1'-4" long
Wood Plug
FLASHBOARD DETAIL
SCALE: 1 IN. = 1 FT., 0 IN.
U bolt
Flow
4" steel pipe, 5' long
Elev. 982.0
SECTION C-C
SCALE: 3/4 IN. = 1 FT., 0 IN.
3" x 10" x 4' wood plank. Fasten into abutment with 3-½" expansion bolts
2-½" steel plates
2" Elbow
Eyebolt set in concrete
2" Double extra strong pipe
4" Oak flashboard
2" Double extra strong pipe, 4'-6¼"
Abutment wall
North end assembly. South end assembly similar but on other hand.
OPERATING ASSEMBLY
SCALE: 1/4 IN. = 1 IN.
See detail for location of operating assembly
Existing South Abutment
Line of dam (upstream)
3" x 10" x 4' long. Fasten into abutment with three ½" expansion bolts.
℄ of pipe
6' long
8' long
4" steel pipe. See detail drawing of U-Bolt connection & joint.
4" oak planks
Note: Two end flashboards 6' long, all others 8' long. 28 sections, 8' long; 2 sections, 6' long ±
Line of dam (downstream)
236' On



CONVERSE WARD DAVIS DIXON
CONSULTING ENGINEERS
PLATE II AUGUST 1978

N.Y.S. DAM NO. 544

REPAIR & RECONSTRUCTION OF WELLS DAM ON THE SACANDAGA RIVER

TOWN OF WELLS, HAMILTON COUNTY, N.Y.

SCALES: AS SHOWN AUGUST, 1949

Morrell Vrooman

MORRELL VROOMAN ENGINEERS
LICENSE NO. 4856

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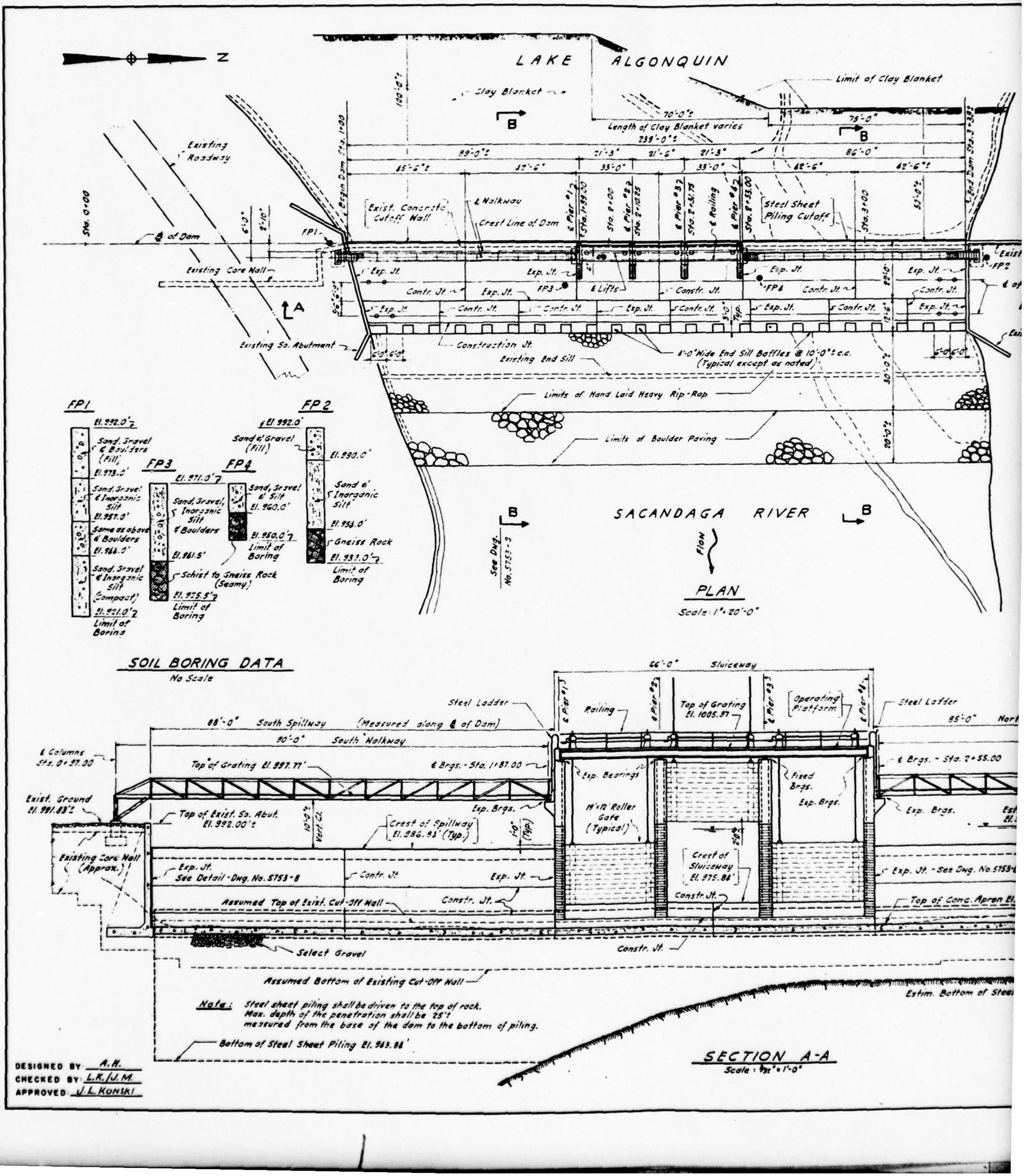
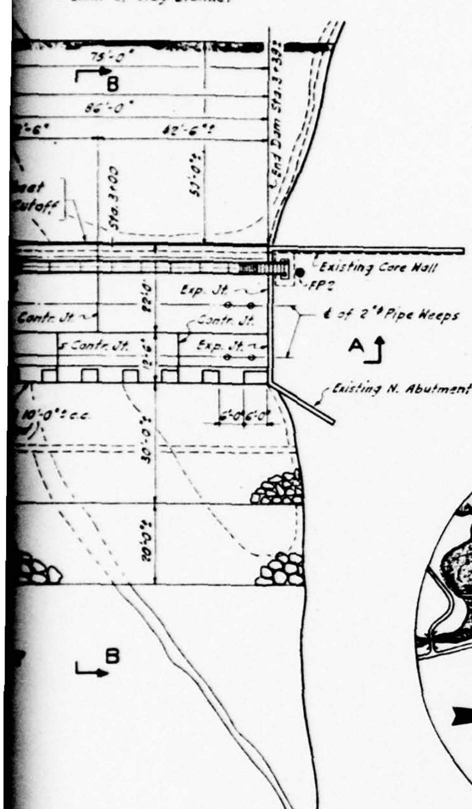
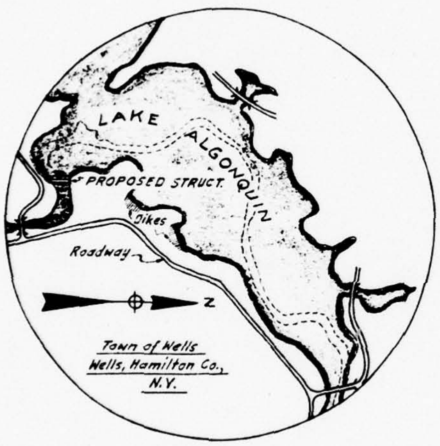


LAKE ALGONQUIN
Limit of Clay Blanket
Clay Blanket
70'-0"±
Length of Clay Blanket varies
239'-0"±
75'-0"
89'-0"±
21'-3"
21'-6"
21'-3"
86'-0"
65'-6"±
32'-6"
33'-0"
33'-0"
62'-6"
62'-6"±
Existing Roadway
Sta. 0+00
Begin Dam Sta. 1+00
Exist. Concrete Cutoff Wall
℄ Walkway
Crest Line of Dam
℄ Pier #1
Sta. 1+88.20
Sta. 2+00
℄ Pier #2
℄ Pier #3
℄ Railing
℄ Pier #4
Steel Sheet Piling Cutoff
Sta. 3+00
End Dam Sta. 3+35
℄ of Dam
FP1
FP2
FP3
FP4
Existing Core Wall
Exp. Jt.
Contr. Jt.
Constr. Jt.
℄ Lifts
Existing So. Abutment
Construction Jt.
Existing End Sill
6'-0" Wide End Sill Baffles @ 10'-0"± c.c. (Typical except as noted)
Limits of Hand Laid Heavy Rip-Rap
Limits of Boulder Paving
SACANDAGA RIVER
Flow
See Dwg. No. 5753-3
PLAN
Scale: 1" = 20'-0"
FP1
El. 992.0'
Sand, Gravel & Boulders (Fill)
El. 973.0'
Sand, Gravel & Inorganic Silt
El. 957.0'
Same as above & Boulders
El. 946.0'
Sand, Gravel & Inorganic Silt (Compact)
El. 921.0'
Limit of Boring
FP3
El. 971.0'
Sand, Gravel, Inorganic Silt & Boulders
El. 961.5'
Schist to Gneiss Rock (Seamy)
El. 925.5'
Limit of Boring
FP4
Sand, Gravel & Silt
El. 960.0'
El. 950.0'
Limit of Boring
FP2
El. 992.0'
Sand & Gravel (Fill)
El. 980.0'
Sand & Inorganic Silt
El. 956.0'
Gneiss Rock
El. 937.0'
Limit of Boring
SOIL BORING DATA
No Scale
66'-0" Sluiceway
Steel Ladder
Railing
Top of Grating El. 1005.97
Operating Platform
88'-0" South Spillway (Measured along ℄ of Dam)
90'-0" South Walkway
85'-0" North
℄ Columns Sta. 0+97.00
Top of Grating El. 997.77'
℄ Brgs. - Sta. 1+87.00
℄ Brgs. - Sta. 2+55.00
Exp. Bearings
Fixed Brgs.
Exp. Brgs.
Exist. Ground El. 991.83'±
Top of Exist. So. Abut. El. 992.00'±
10'-0" Vert. Cl.
Crest of Spillway El. 986.33' (Typ.)
1'-0" (Typ.)
19'x12' Roller Gate (Typical)
Existing Core Wall (Approx.)
Exp. Jt. See Detail - Dwg. No. 5753-8
Contr. Jt.
Exp. Jt.
Crest of Sluiceway El. 975.88'
Exp. Jt. - See Dwg. No. 5753-
Assumed Top of Exist. Cut-Off Wall
Constr. Jt.
Top of Conc. Apron El.
Select Gravel
Assumed Bottom of Existing Cut-Off Wall
Estim. Bottom of Steel
Note: Steel sheet piling shall be driven to the top of rock. Max. depth of the penetration shall be 25'± measured from the base of the dam to the bottom of piling.
Bottom of Steel Sheet Piling El. 963.88'
SECTION A-A
DESIGNED BY: A.H.
CHECKED BY: L.K./J.M.
APPROVED: J.L. KONSKI



NO.	DESCRIPTION	UNIT	QUANTITY
1	Excavation, Unclassified	C.Y.	4 195
2	Steel Sheet Piling Cut-Off	S.F.	5 130
3	Cofferdam & Unwatering	L.S.	Nec.
4	Coarse Sand for Filters	C.Y.	60
5	Select Gravel	C.Y.	180
6A	Concrete Class A - Piers & Sluiceway	C.Y.	313
6B	Concrete Class B - Spillways	C.Y.	1340
6C	Concrete Class B - Apron, End Sill & Foundations	C.Y.	208
7	Reinforcing Steel	Lb.	56 815
8	Rip-Rap	C.Y.	515
9	Boulder Paving, Dumped	C.Y.	250
10	Repair of Existing Abutments	L.S.	Nec.
11	Structural Steel	Lb.	40 140
12	Floor Grating	S.F.	825
13	Installation of Gates	L.S.	Nec.
14	Miscellaneous Metals	L.S.	Nec.
15	Pipe Railing & Ladders	L.S.	Nec.
16	Upstream Clay Blanket	C.Y.	2 310

Caption: ESTIMATE OF QUANTITIES



LOCATION PLAN

Scale: 1" = 1200'

GENERAL NOTES:

All bar reinforcement shall be deformed type, conforming to A.S.T.M. Specifications A305-56T.

All billets for bar reinforcement shall conform to A.S.T.M. Specifications A15-56T.

Reinforcing bars shall not be spliced at places other than shown on the drawings unless approved by the Engineer.

No bar reinforcement shall extend through any expansion or contraction joint.

All structural steel shall conform to the requirements of Specifications for Structural Steel for Welding, A.S.T.M. Designation A373.

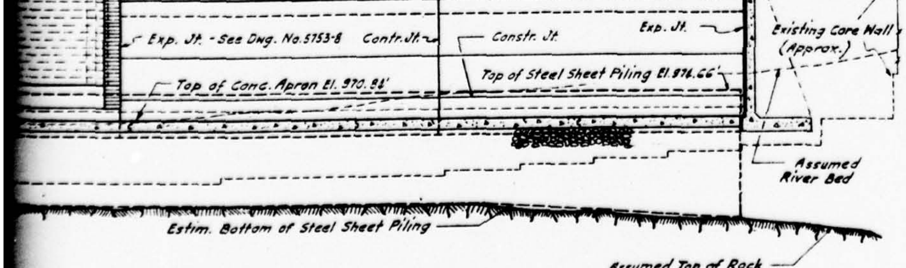
All welding shall comply with the current Specifications for Welded Highway and Railway Bridges - Design, Construction and Repair of the A.W.S., using approved electrodes.

No construction joints other than those shown on the drawings shall be approved.

No steel surfaces embedded in concrete shall be painted.

Joints in the steel sheet piling shall be aligned with the expansion joints adjacent to the piers.

All concrete shall comply with the Specifications.



-A

CONVERSE WARD DAVIS DIXON
CONSULTING ENGINEERS
PLATE III AUGUST 1978

PREPARED
ERDMAN, ANTHONY & HOSLEY, CONSULTING ENGINEERS
N.Y.S.P.E. LIC. NO. 27677 DATE: JULY 26, 1958

TOWN OF WELLS
WELLS, HAMILTON COUNTY, N.Y.

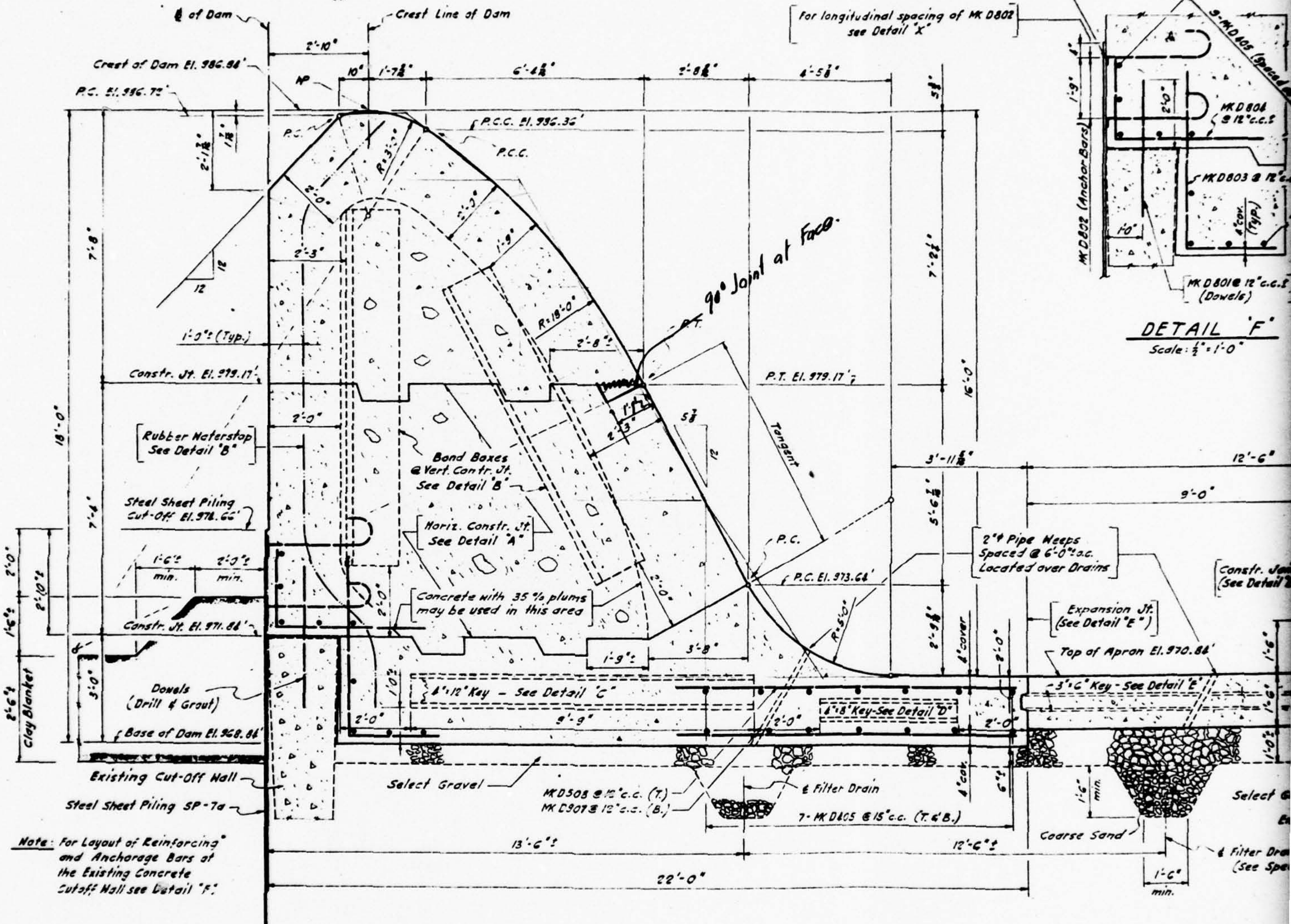
ERDMAN, ANTHONY & HOSLEY
ROCHESTER, NEW YORK

LAKE ALGONQUIN DAM

GENERAL PLAN

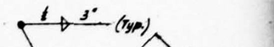
DATE: JUNE 19, 1958 SHEET 1 OF 8 DWG. NO. 5733-1

2



SECTION B-B

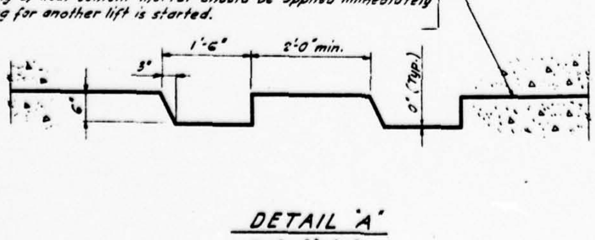
Scale: ½" = 1'-0"



DETAIL "F"

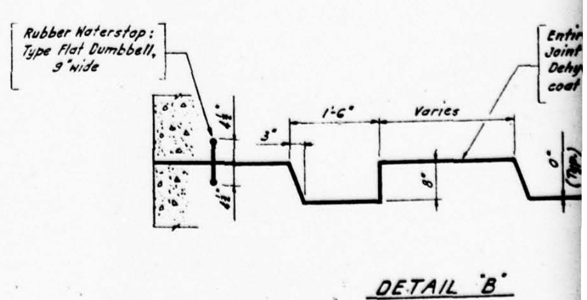
Scale: ½" = 1'-0"

Note: Entire Surface of Horizontal Construction Joint shall be thoroughly cleaned of laitance and other objectionable material in a manner satisfactory to the Engineer. A ½" thin coating of neat cement mortar should be applied immediately before concrete placing for another lift is started.



DETAIL "A"

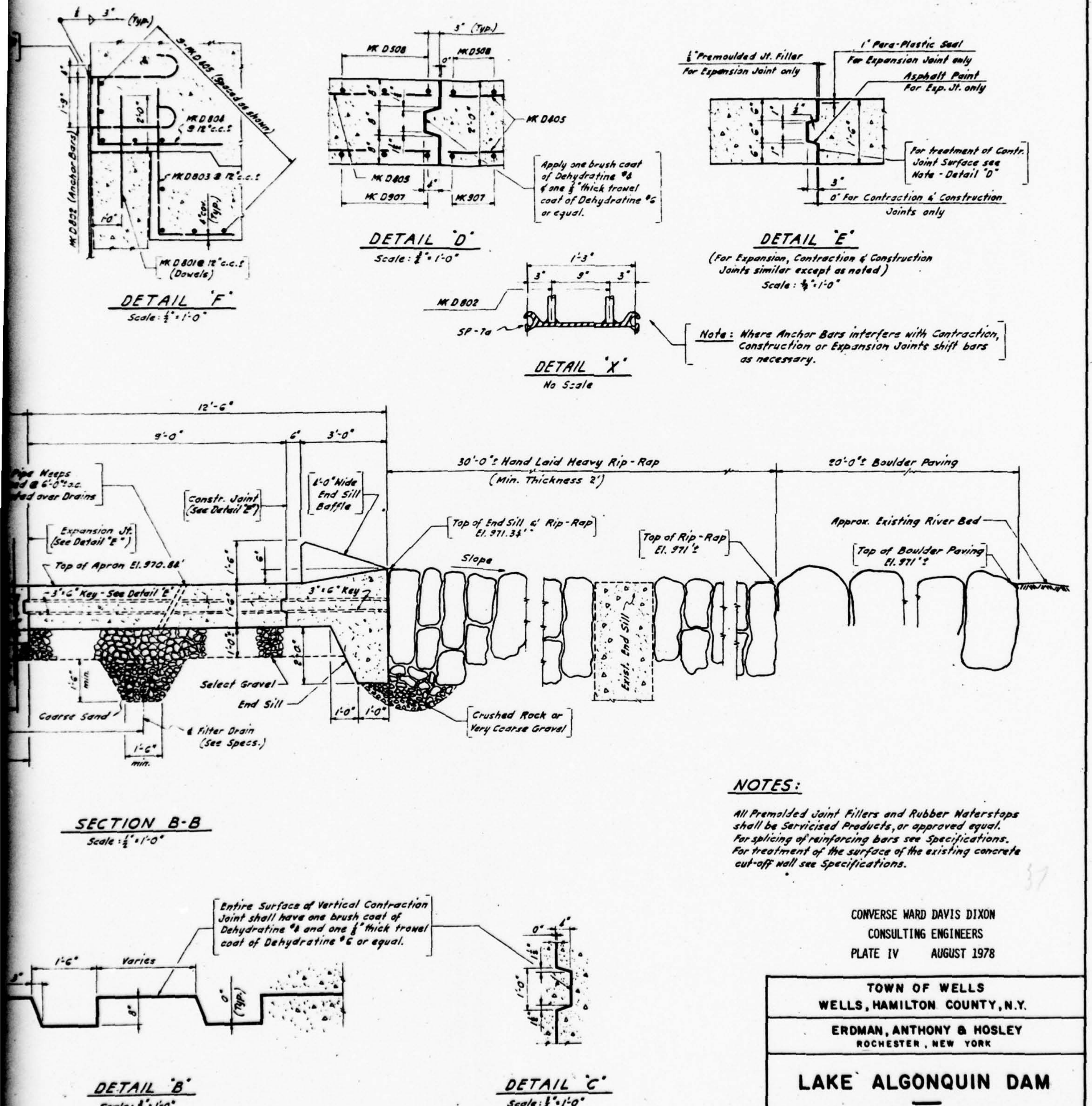
Scale: ⅜" = 1'-0"



DETAIL "B"

Scale: ⅜" = 1'-0"

DESIGNED BY: A. H.
CHECKED BY: L.K./J.M.
APPROVED: J.L. KONSKI



NOTES:

All Premolded Joint Fillers and Rubber Waterstops shall be Servicised Products, or approved equal.
For splicing of reinforcing bars see Specifications.
For treatment of the surface of the existing concrete cut-off wall see Specifications.

CONVERSE WARD DAVIS DIXON
CONSULTING ENGINEERS
PLATE IV AUGUST 1978

TOWN OF WELLS
WELLS, HAMILTON COUNTY, N.Y.

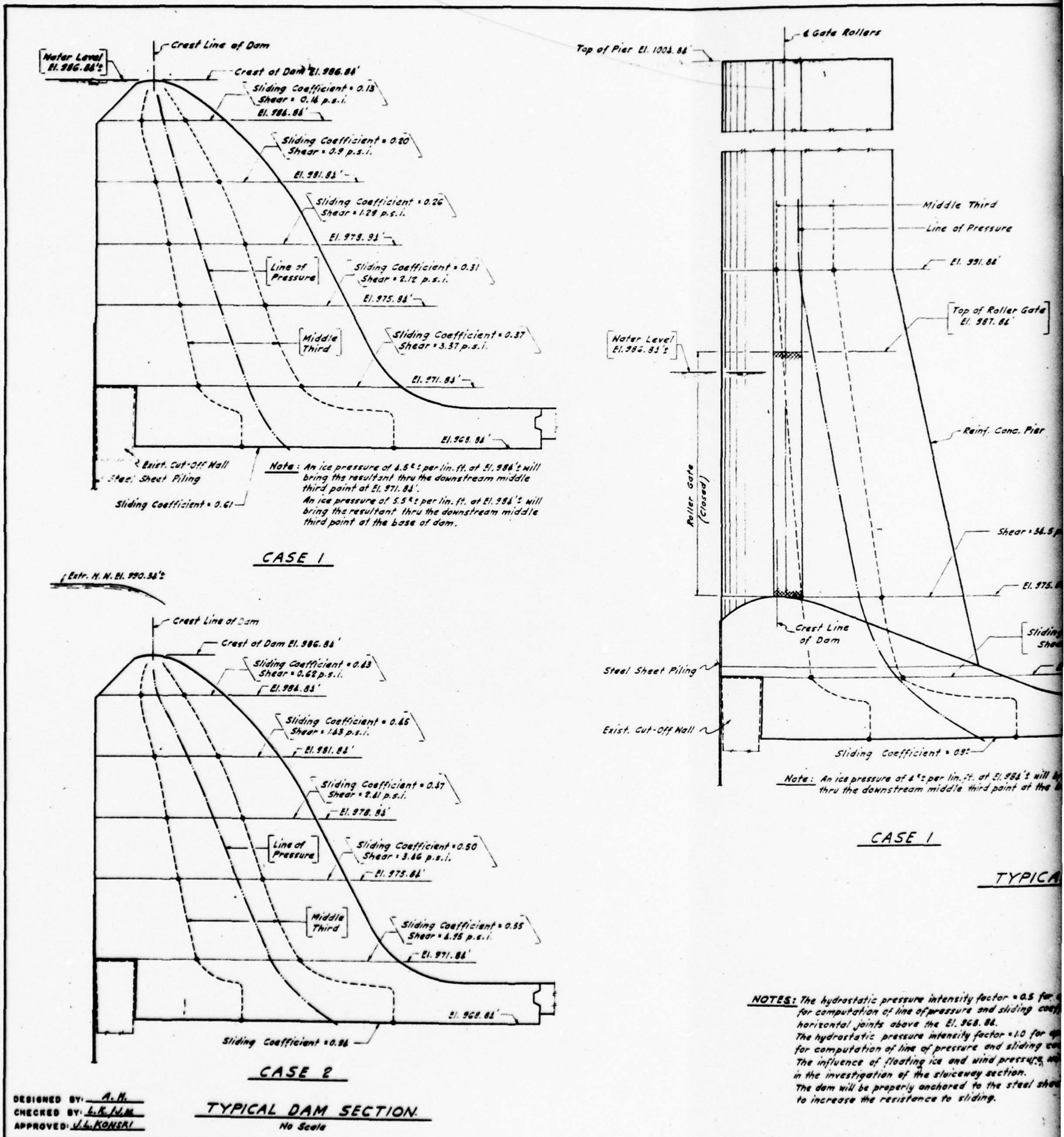
ERDMAN, ANTHONY & HOSLEY
ROCHESTER, NEW YORK

LAKE ALGONQUIN DAM

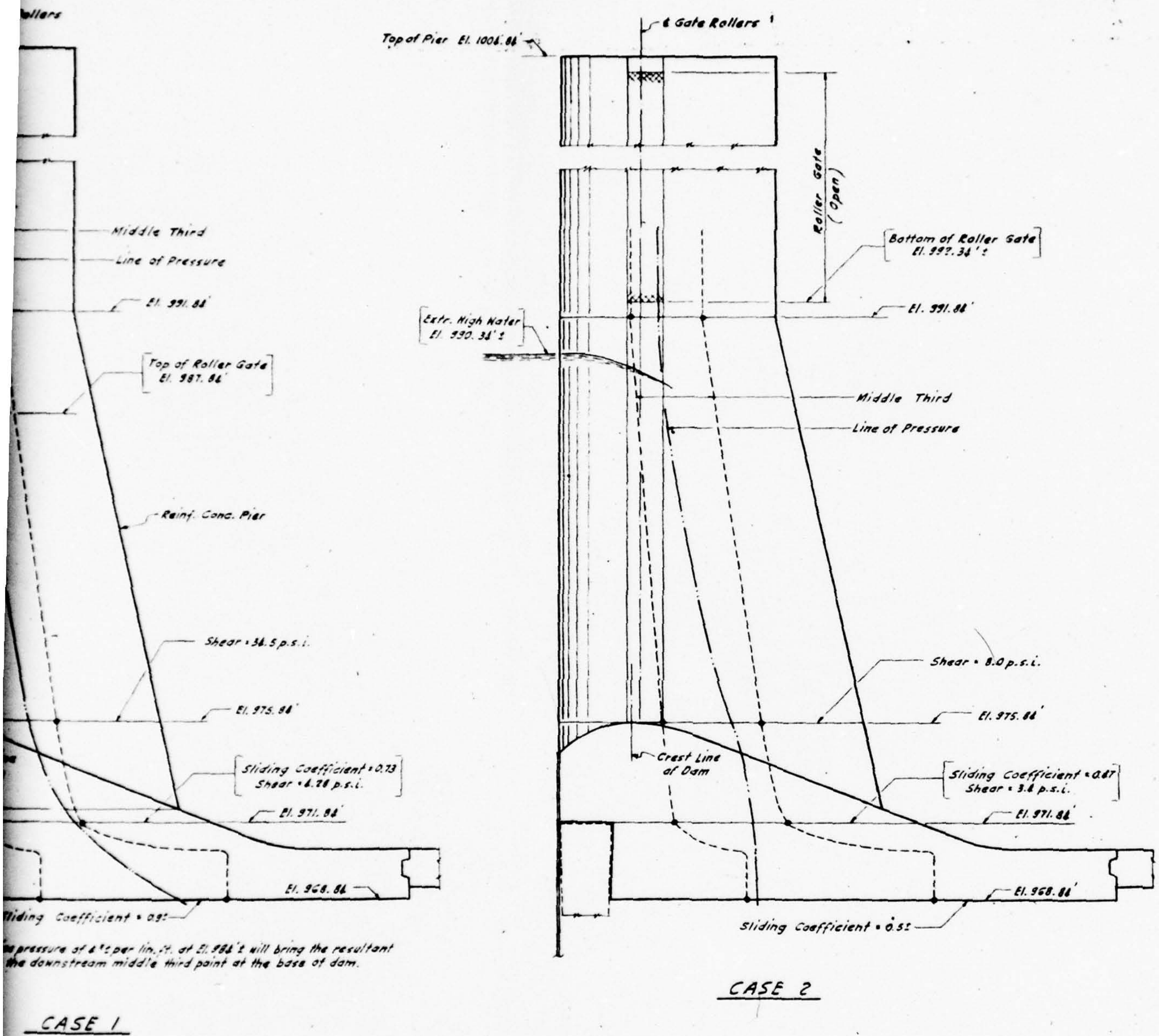
SPILLWAY CROSS-SECTION

DATE: JUNE 19, 1958 SHEET 2 OF 8 DWG. NO. 5758-2

PREPARED N.Y.S.P.E. LIC. NO. 27677 DATE: JULY 26, 1956
ERDMAN, ANTHONY & HOSLEY, CONSULTING ENGINEERS



Water Level El. 986.86'±
Crest Line of Dam
Crest of Dam El. 986.86'
Sliding Coefficient = 0.13
Shear = 0.16 p.s.i.
El. 984.86'
Sliding Coefficient = 0.20
Shear = 0.9 p.s.i.
El. 981.86'
Sliding Coefficient = 0.26
Shear = 1.29 p.s.i.
El. 978.96'
Line of Pressure
Sliding Coefficient = 0.31
Shear = 2.12 p.s.i.
El. 975.86'
Middle Third
Sliding Coefficient = 0.37
Shear = 3.37 p.s.i.
El. 971.86'
El. 968.86'
Exist. Cut-Off Wall
Steel Sheet Piling
Sliding Coefficient = 0.61
Note: An ice pressure of 6.5K± per lin. ft. at El. 986'± will bring the resultant thru the downstream middle third point at El. 971.86'.
An ice pressure of 5.5K± per lin. ft. at El. 986'± will bring the resultant thru the downstream middle third point at the base of dam.
CASE 1
Extr. H. W. El. 990.36'±
Crest Line of Dam
Crest of Dam El. 986.86'
Sliding Coefficient = 0.63
Shear = 0.62 p.s.i.
El. 984.86'
Sliding Coefficient = 0.65
Shear = 1.63 p.s.i.
El. 981.86'
Sliding Coefficient = 0.57
Shear = 2.61 p.s.i.
El. 978.86'
Line of Pressure
Sliding Coefficient = 0.50
Shear = 3.46 p.s.i.
El. 975.86'
Middle Third
Sliding Coefficient = 0.55
Shear = 6.95 p.s.i.
El. 971.86'
El. 968.86'
Sliding Coefficient = 0.86
CASE 2
DESIGNED BY: A. H.
CHECKED BY: L.K./J.M.
APPROVED: J.L. KONSKI
TYPICAL DAM SECTION
No Scale
℄ Gate Rollers
Top of Pier El. 1006.86'
Middle Third
Line of Pressure
El. 991.86'
Top of Roller Gate El. 987.86'
Water Level El. 986.86'±
Reinf. Conc. Pier
Roller Gate (Closed)
Shear = 36.5 p
El. 975.
Crest Line of Dam
Steel Sheet Piling
Sliding
Shea
Exist. Cut-Off Wall
Sliding Coefficient = 0.9
Note: An ice pressure of 6K± per lin. ft. at El. 986'± will b
thru the downstream middle third point at the b
CASE 1
TYPICA
NOTES: The hydrostatic pressure intensity factor = 0.5 for
for computation of line of pressure and sliding coeff
horizontal joints above the El. 968.86.
The hydrostatic pressure intensity factor = 1.0 for
for computation of line of pressure and sliding co
The influence of floating ice and wind pressure
in the investigation of the sluiceway section.
The dam will be properly anchored to the steel shee
to increase the resistance to sliding.



TYPICAL SLUICEWAY SECTION

No Scale

static pressure intensity factor = 0.5 for uplift will be used
tation of line of pressure and sliding coefficients for
t joints above the El. 968.86.
tatic pressure intensity factor = 1.0 for uplift will be used
utation of line of pressure and sliding coefficients for the base of dam.
nce of floating ice and wind pressure will be neglected
stigation of the sluiceway section.
ill be properly anchored to the steel sheet piling cutoff
se the resistance to sliding.

CONVERSE WARD DAVIS DIXON
CONSULTING ENGINEERS
PLATE V AUGUST 1978

TOWN OF WELLS
WELLS, HAMILTON COUNTY, N.Y.

ERDMAN, ANTHONY & HOSLEY
ROCHESTER, NEW YORK

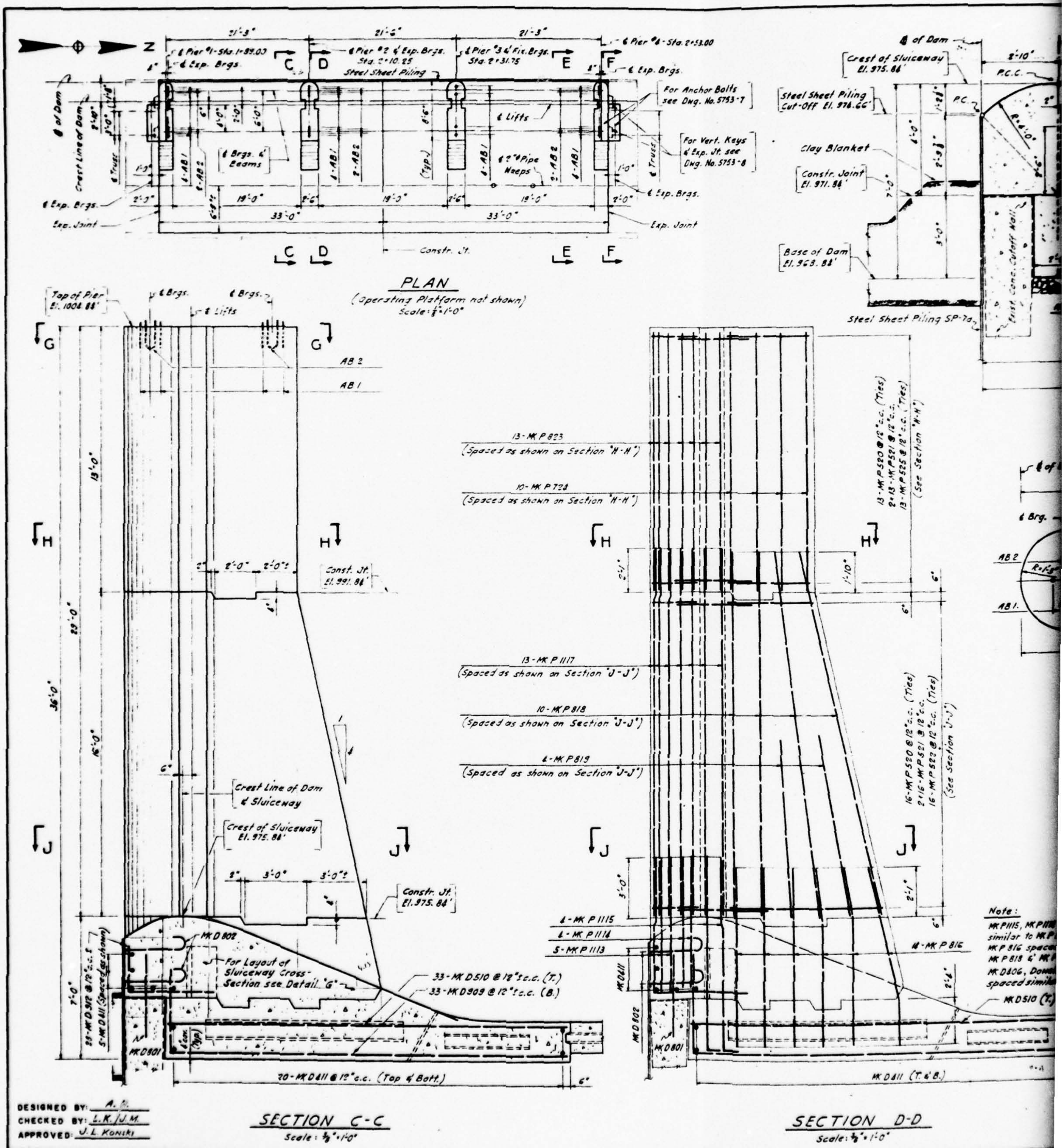
LAKE ALGONQUIN DAM

STABILITY DIAGRAMS

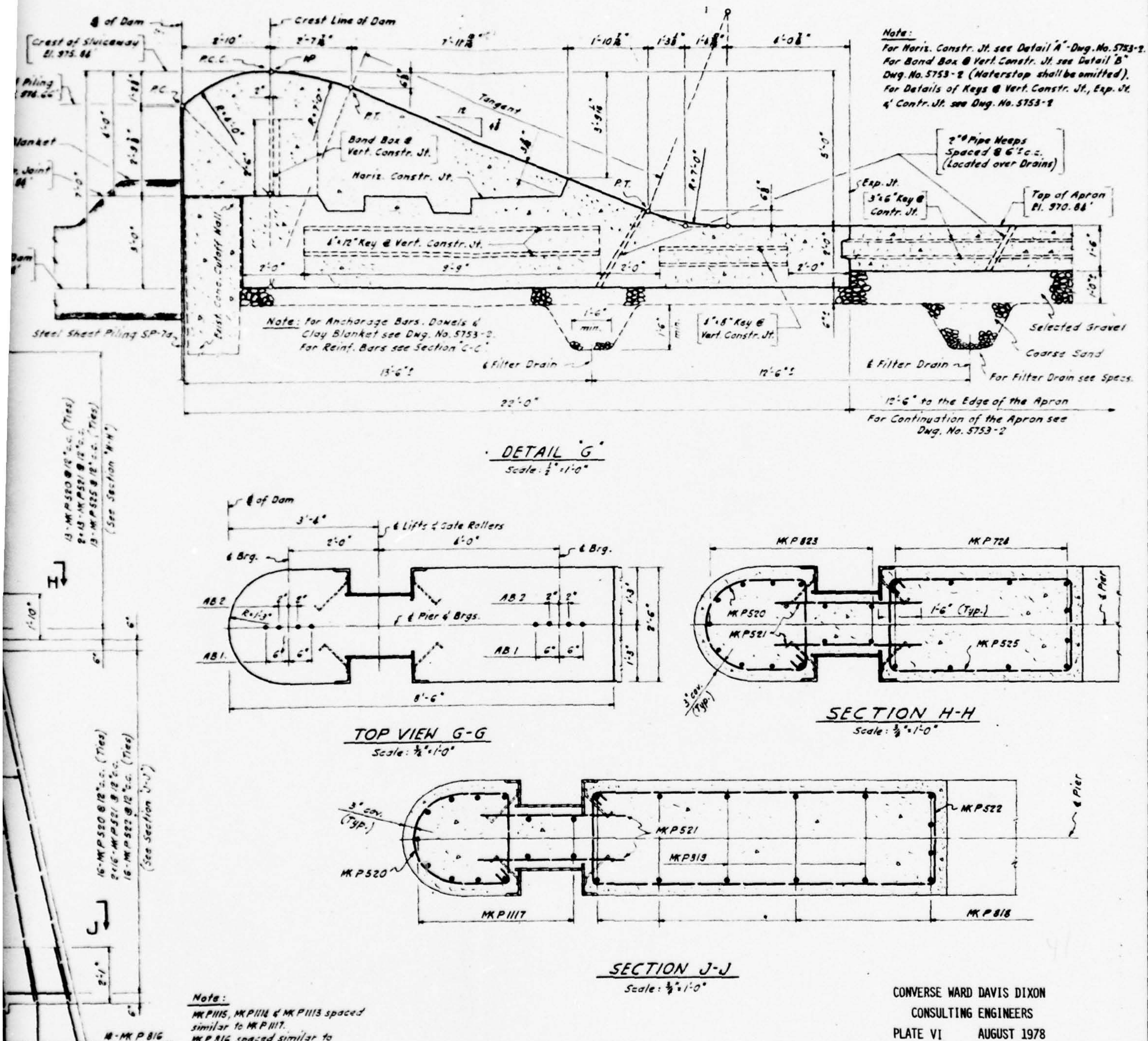
PREPARED
ERDMAN, ANTHONY & HOSLEY, CONSULTING ENGINEERS N.Y.S.P.E. LIC. NO. 27677 DATE: JULY 26, 1958

DATE: JUNE 19, 1958 SHEET 3 OF 8 DWG. NO. 5793-3

2



PLAN
(Operating Platform not shown)
Scale: 1/8" = 1'-0"
℄ Pier #1 - Sta. 1+89.00
℄ Exp. Brgs.
℄ Pier #2 ℄ Exp. Brgs. Sta. 2+10.25
Steel Sheet Piling
℄ Pier #3 ℄ Fix. Brgs. Sta. 2+31.75
℄ Pier #4 - Sta. 2+53.00
℄ Exp. Brgs.
For Anchor Bolts see Dwg. No. 5753-7
For Vert. Keys ℄ Exp. Jt. see Dwg. No. 5753-8
℄ Lifts
℄ Brgs. ℄ Beams
℄ 2" ϕ Pipe Weeps
Crest Line of Dam
℄ of Dam
℄ Truss
Exp. Joint
Constr. Jt.
21'-3"
21'-6"
21'-3"
19'-0"
33'-0"
℄ of Dam
Crest of Sluiceway El. 975.86'
Steel Sheet Piling Cut-Off El. 976.66'
Clay Blanket
Constr. Joint El. 971.86'
Base of Dam El. 963.86'
Steel Sheet Piling SP-7a
P.C.C.
Exist. Conc. Cutoff Wall
SECTION C-C
Scale: 3/8" = 1'-0"
Top of Pier El. 1004.86'
℄ Brgs.
℄ Lifts
AB 2
AB 1
Const. Jt. El. 991.86'
Crest Line of Dam ℄ Sluiceway
Crest of Sluiceway El. 975.86'
Constr. Jt. El. 975.86'
MK D802
For Layout of Sluiceway Cross-Section see Detail "G"
33-MK D510 @ 12" ± c.c. (T.)
33-MK D909 @ 12" ± c.c. (B.)
MK D801
20-MK D611 @ 12" c.c. (Top ℄ Bott.)
13'-0"
23'-0"
36'-0"
16'-0"
7'-0"
SECTION D-D
Scale: 3/8" = 1'-0"
13-MK P823 (Spaced as shown on Section "H-H")
10-MK P728 (Spaced as shown on Section "H-H")
13-MK P1117 (Spaced as shown on Section "J-J")
10-MK P818 (Spaced as shown on Section "J-J")
6-MK P819 (Spaced as shown on Section "J-J")
13-MK P520 @ 12" c.c. (Ties)
2×13-MK P521 @ 12" c.c.
13-MK P525 @ 12" c.c. (Ties)
(See Section "H-H")
16-MK P520 @ 12" c.c. (Ties)
2×16-MK P521 @ 12" c.c.
16-MK P522 @ 12" c.c. (Ties)
(See Section "J-J")
6-MK P1115
6-MK P1116
5-MK P1113
14-MK P816
MK D611
MK D802
MK D801
MK D611 (T. ℄ B.)
MK D510 (T.)
Note:
MK P1115, MK P1116 ... similar to MK P...
MK P816 spaced ...
MK P818 ℄ MK P...
MK D806, Dowels ... spaced similar...
DESIGNED BY: A. M.
CHECKED BY: L.K./J.M.
APPROVED: J. L. Konski



CONVERSE WARD DAVIS DIXON
CONSULTING ENGINEERS
PLATE VI AUGUST 1978

TOWN OF WELLS
WELLS, HAMILTON COUNTY, N.Y.

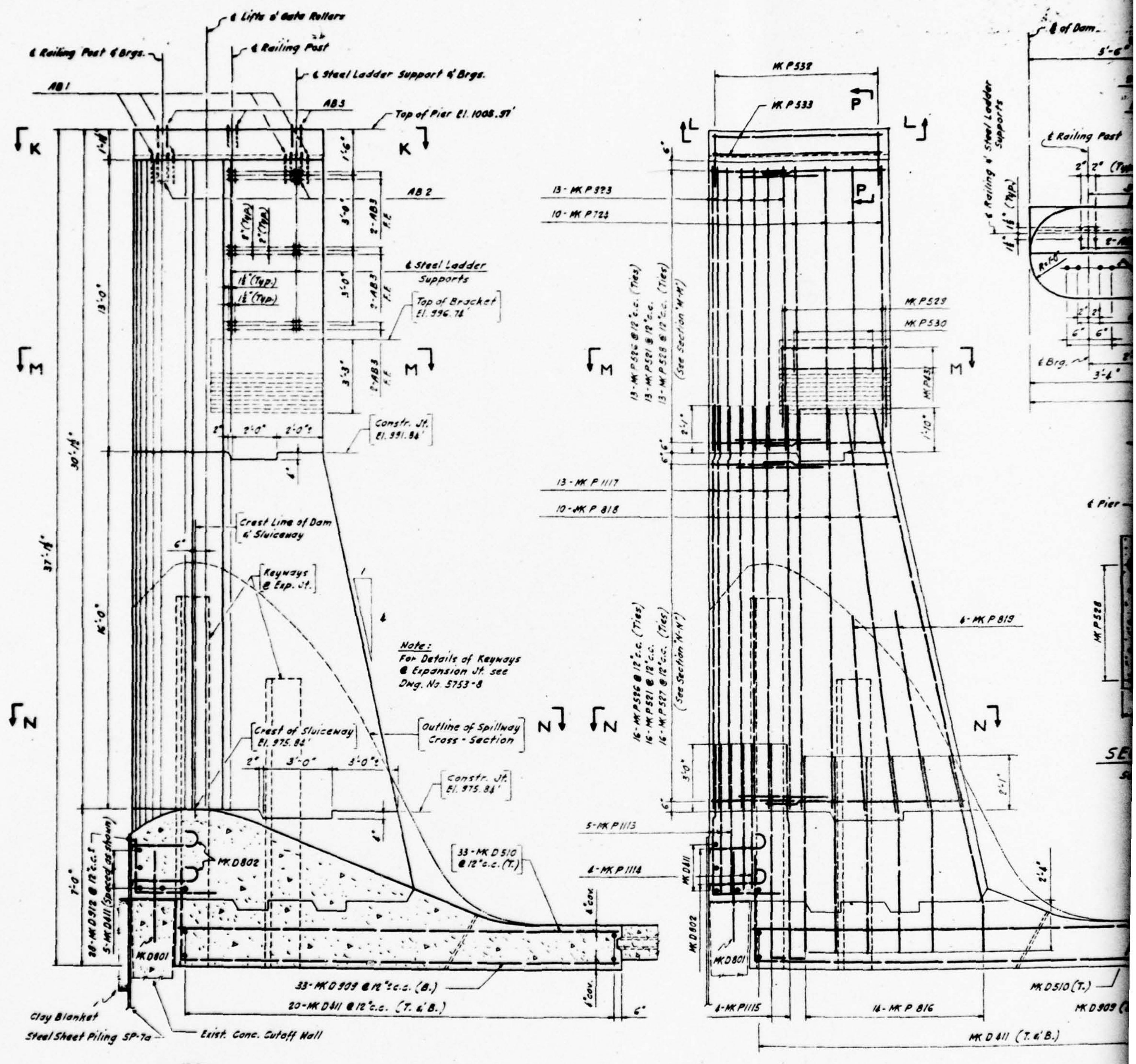
ERDMAN, ANTHONY & HOSLEY
ROCHESTER, NEW YORK

LAKE ALGONQUIN DAM

PIER DETAILS

PREPARED [signature] N.Y.S.P.E. LIC. NO. 27677 DATE JULY 26, 1958
ERDMAN, ANTHONY & HOSLEY, CONSULTING ENGINEERS

DATE JUNE 19, 1958 SHEET 4 OF 8 DWG. NO. 5753-4



Note:
For Layout of Sluiceway Cross-Section see Detail "G" - Dwg. No. 5753-4.
For Anchorage Bars, Dowels & Clay Blanket see Dwg. No. 5753-2.

SECTION E-E

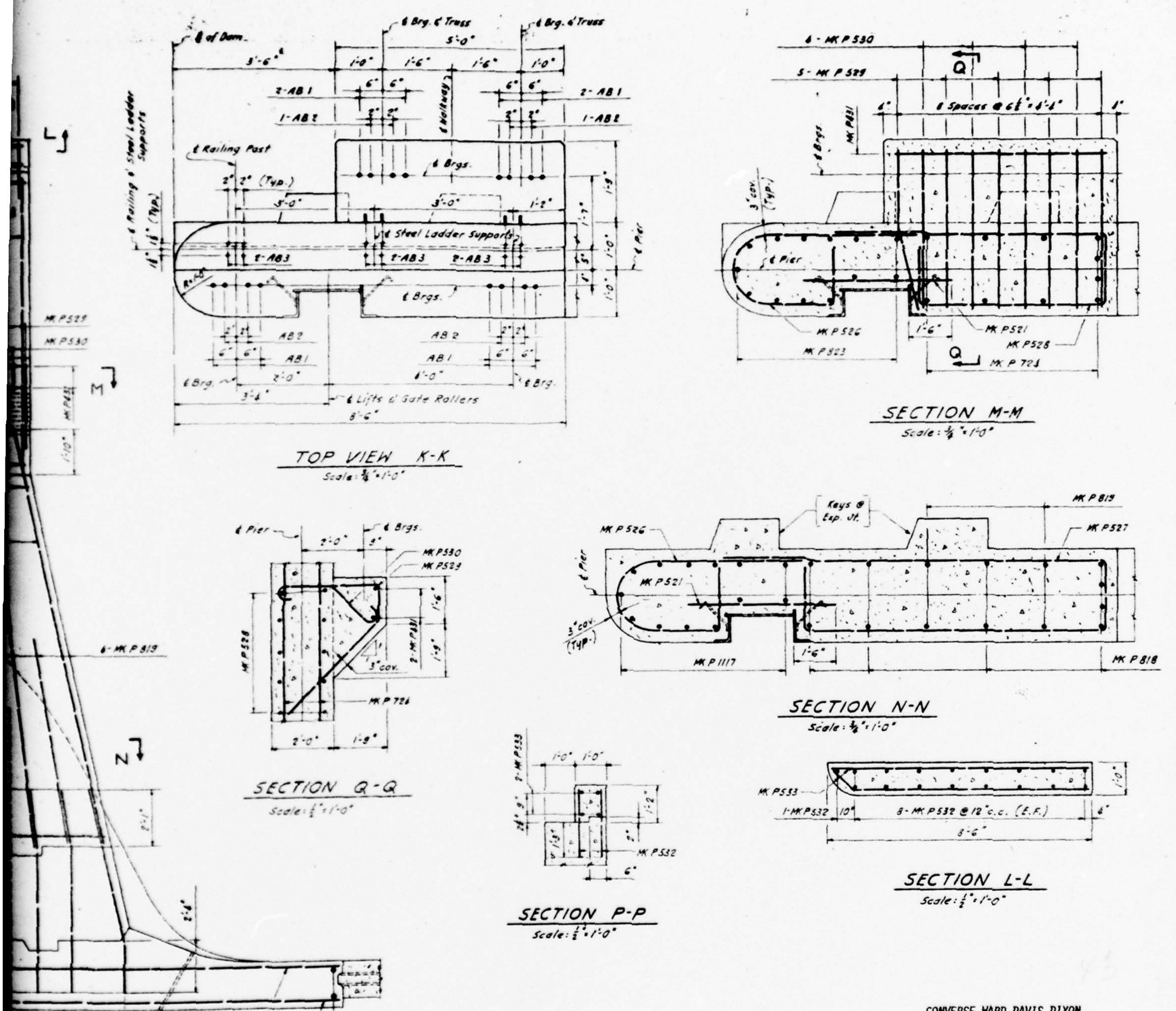
Scale: 3/8" = 1'-0"

Note:
For spacing of MK P823 & MK P724 see Section "M-M."
For spacing of MK P1117, MK P818 & MK P819 see Section "N-N."
MK P1113, MK P1114 & MK P1115 spaced similar to MK P1117.
MK P816 spaced similar to MK P818 & MK P819.

SECTION F-F

Scale: 3/8" = 1'-0"

DESIGNED BY: A.H.
CHECKED BY: L.K./J.M.
APPROVED: J.L.KONSKI



CONVERSE WARD DAVIS DIXON
CONSULTING ENGINEERS
PLATE VII AUGUST 1978

NOTES: Pier No. 1 similar to Pier No. 4 but opposite hand.
Steel housing for gate rollers furnished by Armco Drainage and Metal Products, Inc.
Installation included in Concrete item.

...& MK P 726 see Section "M-M".
..., MK P 818 & MK P 819 see Section "N-N".
...P 1115 spaced similar to MK P 1117.
... to MK P 818 & MK P 819.

...TION F-F
...: 3/8" = 1'-0"

TOWN OF WELLS
WELLS, HAMILTON COUNTY, N.Y.

ERDMAN, ANTHONY & HOSLEY
ROCHESTER, NEW YORK

LAKE ALGONQUIN DAM

PIER DETAILS

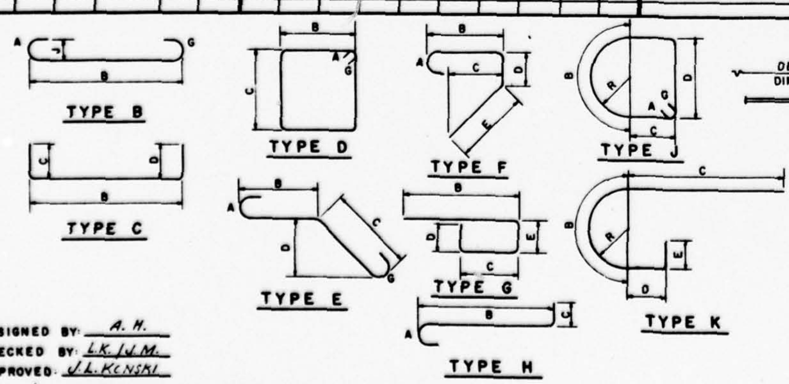
PREPARED N.Y.S.P.E. LIC. NO. 27677 DATE JULY 26, 1958
ERDMAN, ANTHONY & HOSLEY, CONSULTING ENGINEERS

DATE: JUNE 19, 1958 SHEET 5 OF 8 DWG. NO. 5753-5

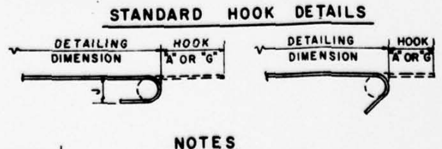
2

REINFORCING SCHEDULE

MARK	SIZE	NO.	LENGTH	TYPE	WT.	A	B	C	D	E	F	G	H	J	R	LOCATION
D801	#8	240	6'-0"	A	2563											Dowels in Exist. Cutoff Wall
D802	#8	768	6'-4"	H	8979	1'-1"	3'-0"	0'-3"								Anchor Bars for Sheet Piling
D803	#9	173	7'-5"	C	3457		2'-6"	6'-11"	0							Corner Bars at Exist. Cutoff Wall
D804	#8	176	6'-6"	C	3070		4'-3"	2'-3"	0							-"- -"- -"- -"-
D405	#4	92	62'-0"	A	2561											Spillway - Longitudinal - T.&B.
D406	#4	54	4'-6"	A	162											Sluiceway Dowels & Spillway at So. Abut.
D907	#9	169	10'-0"	A	5766											Spillway - Transverse - Bott.
D508	#5	169	10'-0"	A	1763											-"- -"- Top
D909	#9	66	25'-5"	C	5704		19'-6"	6'-11"	0							Sluiceway - Transverse - Bott.
D510	#5	66	19'-2"	A	1332											-"- -"- Top
D411	#4	90	32'-6"	A	1956											-"- Longitudinal - T.&B.
D912	#9	56	6'-6"	C	1238		4'-3"	2'-3"	0							-"- Corner at Exist. Cutoff
P1113	#11	20	11'-8"	C	1239		6'-8"	6'-8"	0							Pier #1, 2, 3 & 4 - Dowels
P1114	#11	16	6'-8"	A	567											-"- -"-
P1115	#11	16	10'-7"	C	899		1'-1"	9'-6"	0							-"- -"-
P816	#8	56	8'-5"	A	1259											-"- -"-
P1117	#11	52	18'-1"	A	4995											-"- Vertical
P818	#8	40	17'-10"	A	1904											-"- -"-
P819	#8	16	8'-0"	A	342											-"- -"-
P520	#5	59	8'-2"	J	504	0'-5"	3'-2"	1'-2"	2'-0"			0'-5"			1'-0"	Pier #2 & 3 - Ties
P521	#5	178	4'-4"	A	796											Pier #1, 2, 3 & 4 - Horizontal
P522	#5	32	18'-3" to 20'-6"	D	559	0'-5"	[illegible] to 7'-10"	3'-0"	(Varies with Incr. 3"±)			0'-5"				Pier #2 & 3 - Ties
P823	#8	52	12'-9"	A	1770											Pier #1, 2, 3 & 4 - Vertical
P724	#7	60	12'-9"	A	1562											-"- -"-
P525	#5	26	12'-10"	D	348	0'-5"	4'-0"	2'-0"				0'-5"				Pier #2 & 3 - Ties
P526	#5	59	8'-9"	K	539		2'-4"	3'-6"	1'-5"	1'-6"					0'-9"	Pier #1 & 4 - Ties
P527	#5	32	18'-9" to 20'-3"	G	551		[illegible]	[illegible] to 7'-10"	1'-6"	1'-6"	(Varies with Incr. 3"±)					-"- -"-
P528	#5	26	12'-7"	G	341		5'-7"	4'-0"	1'-6"	1'-6"						-"- -"-
P529	#5	10	11'-9"	F	123	0'-7"	3'-3"	2'-10"	1'-0"	4'-1"						-"- Bracket
P530	#5	8	5'-8"	E	47	0'-7"	1'-7"	1'-10"	1'-1"			0'-7"				-"- -"-
P431	#4	4	4'-6"	A	12											-"- -"-
P532	#5	[illegible]	8'-3"	A	80											-"- Dowels @ Top
P533	#5	8	8'-0"	A	67											-"- Horizontal @ Top
								Total Wt. = 56,371#								
FOOTINGS:																
F501	#5	16	4'-6"	A	75											Footing - Transverse - Bott.
F402	#4	16	6'-6"	A	69											-"- -"- Top
F503	#5	10	7'-6"	A	78											-"- Longitudinal - Bott.
F404	#4	10	7'-6"	A	50											-"- -"- Top
F505	#5	23	5'-3"	C	125		0'-6"	4'-6"	0							-"- Vertical
F406	#4	3	12'-9"	D	26	0'-4½"	1'-6"	4'-6"				0'-4½"				-"- Ties
								Total Wt. = 844#								



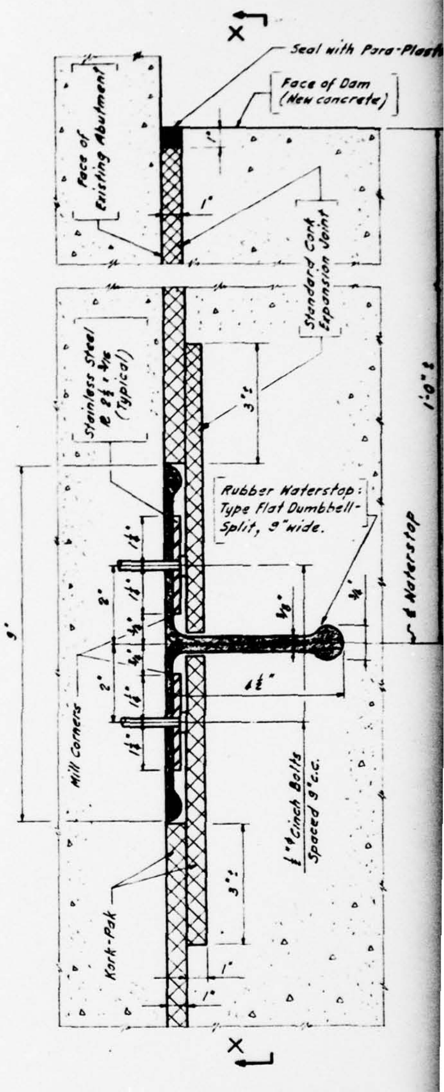
STANDARD HOOK DETAILS



NOTES

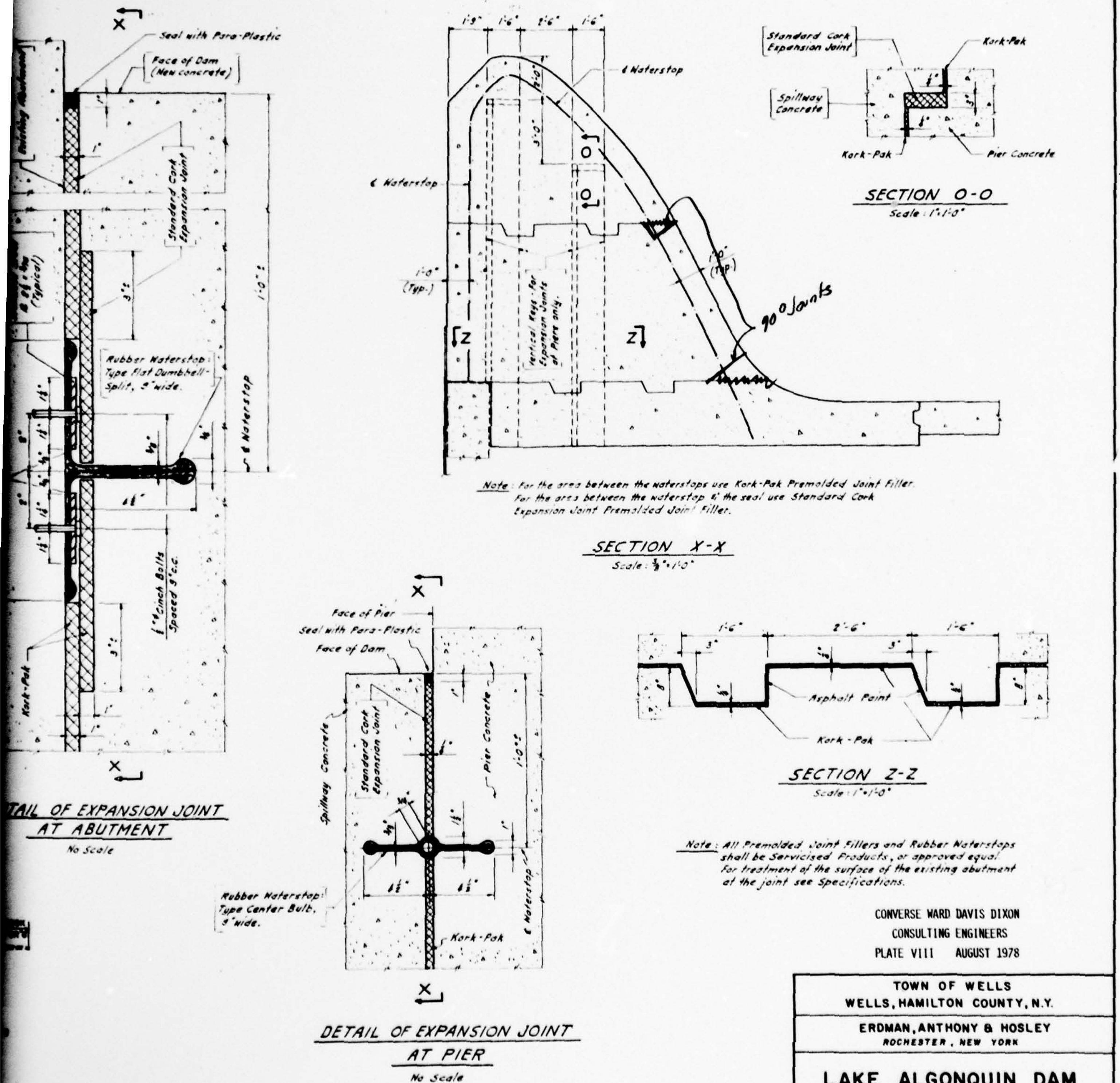
1. ALL STRAIGHT BARS ARE TYPE A.
2. ALL DIMENSIONS ARE MEASURED ALONG OUTSIDE FACE OR OUT TO OUT OF BARS.

DESIGNED BY: A. H.
CHECKED BY: L.K. / J.M.
APPROVED: J.L. KONSKI



DETAIL OF EXPANSION JOINT AT ABUTMENT

No Scale



CONVERSE WARD DAVIS DIXON
CONSULTING ENGINEERS
PLATE VIII AUGUST 1978

TOWN OF WELLS
WELLS, HAMILTON COUNTY, N.Y.

ERDMAN, ANTHONY & HOSLEY
ROCHESTER, NEW YORK

LAKE ALGONQUIN DAM

MISCELLANEOUS DETAILS

PREPARED [signature] N.Y.S.P.E. LIC NO. 27677 DATE: JULY 26, 1956
ERDMAN, ANTHONY & HOSLEY, CONSULTING ENGINEERS

DATE: JUNE 19, 1956 SHEET 8 OF 8 DWG NO. 5753-8

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APPENDIX A

CHECKLIST - ENGINEERING DATA

CHECKLIST

HYDROLOGIC AND HYDRAULIC DATA

ENGINEERING DATA

NAME OF DAM: Lake Algonquin Dam NDS ID NO.: NY 172

RATED CAPACITY (ACRE-FEET) 1200 NYS DEC ID NO.: 171-2700

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 986.84

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): 986.84

ELEVATION MAXIMUM DESIGN POOL: 992.0

ELEVATION TOP DAM: 992.0

CREST (SPILLWAY)

a. Elevation 986.84
b. Type Concrete; ogee type
c. Width Not applicable; crest is rounded
d. Length 235± feet; see below
e. Location Spillover To right and left of gate section; see below
f. Number and Type of Gates None

OUTLET WORKS:

a. Type 3 - 12'x19' vertical lift roller gates
b. Location Gate section is near center of dam
c. Entrance inverts 975.84
d. Exit inverts 975.84
e. Emergency draindown facilities These gates are the only emergency draindown facilities.

HYDROMETEOROLOGICAL GAGES:

a. Type None
b. Location None
c. Records None

MAXIMUM NON-DAMAGING DISCHARGE: 17,300 cfs (gates open) is the design discharge as reported in Application for Construction of a Dam, State of New York Department of Public Works (NYSDPW), 31 July 1958.

SPILLWAY LENGTH:
The dam consists of a right spillway section 88 feet long and a left spillway section 90 feet long, separated by an outlet structure 66 feet long. The tops of the three 19' wide outlet gates are 1 foot above spillway elevation. If, during a flood, the gates are not opened, flow will eventually overtop the gates and the effective length of the spillway will become 235± feet.

CHECKLIST

ENGINEERING DATA

DESIGN, CONSTRUCTION, AND OPERATION
PHASE I

NAME OF DAM: Lake Algonquin Dam

NDS ID NO.: NY172 NYS DEC ID NO.: 171-2700

ITEM	REMARKS
DRAWINGS	A set of 8 design drawings are available, all dated 19 June 1958 by Erdman, Anthony and Hosley, Rochester, N.Y. This set includes: General Plan (1), Spillway Cross-section (1), Pier Details (2), Walkway Details (1), Miscellaneous Details (2) and Stability Diagrams (1).
REGIONAL VICINITY MAP	Dam-lake system shown on USGS 15-minute Quadrangle Sheet of Lake Pleasant, N.Y. (N4315/W7415)
CONSTRUCTION HISTORY	No formal history available. Information concerning previous dam at site is contained in drawings and correspondence on file with NYSDEC.
TYPICAL SECTIONS OF DAM	Available on 1958 drawings
HYDROLOGIC/HYDRAULIC DATA	Some available from report on repair and remodeling of the original structure by Morrell and Vrooman Engineers, Gloversville, N.Y., dated August 1949. (Appendix E)

ENGINEERING DATA

ITEM	REMARKS
OUTLETS: Plan Details Constraints Discharge Ratings	Plan and profile of gates available on 1958 drawings. Constraints and discharge ratings are not available.
RAINFALL/RESERVOIR RECORDS	None available
DESIGN REPORTS	None available for present structure. There is a report on the repair and remodeling of the original structure by Morrell and Vrooman Engineers, Gloversville, N.Y., dated August 1949. (Appendix E)
GEOLOGY REPORTS	None available
DESIGN COMPUTATIONS: Hydrology & Hydraulics Dam Stability Seepage Studies Structural	Hydrology and hydraulics: none available. Dam stability: stability diagrams available on 1958 design drawings. No computations available. Seepage studies: none available. Structural: There are some computations performed for the counterfort wingwall.

ENGINEERING DATA

ITEM	REMARKS
MATERIALS INVESTIGATIONS Boring Records Laboratory Field	Logs of 4 borings are included in General Plan of 1958 drawings.
POST-CONSTRUCTION SURVEYS OF DAM	None available
BORROW SOURCES	Not applicable
MONITORING SYSTEMS	None
MODIFICATIONS	None

ENGINEERING DATA

ITEM	REMARKS
HIGH POOL RECORDS	None available; hearsay evidence obtained from local residents.
POST-CONSTRUCTION ENGINEERING STUDIES AND REPORTS	None available
PRIOR ACCIDENTS OR FAILURE OF DAM Description Reports	Abutment walls of original dam were overtopped by flood on 31 Dec. 1948 because flashboards had not been lowered. Details contained in Morrell Vrooman report dated August 1949. (Appendix E)
MAINTENANCE AND OPERATION RECORDS	None available
SPILLWAY: Plan Sections Details	Plans, sections and details available on 1958 drawings

ENGINEERING DATA

ITEM	REMARKS
OPERATING EQUIPMENT: Plans Details	Plans available on 1958 drawings. No mechanical details or operational procedures available.
PREVIOUS INSPECTION Date: Findings	Inspections are performed periodically by NYSDEC. The latest reported one was on 9 Sept. 1970: "Dam in good condition except for slight amount of bank erosion at wingwalls."

APPENDIX B

CHECKLIST - VISUAL DATA

CHECKLIST

VISUAL INSPECTION

PHASE I

NAME OF DAM: Lake Algonquin Dam County: Hamilton State: New York NDS ID No.: NY 172

Sacandaga River NYS DEC ID No.: 171-2700

Type of Dam: Concrete Gravity Hazard Category: High

Date(s) Inspection: 19 July 1978 Weather: Clear and warm Temperature: 85°F

Pool Elevation at Time of Inspection: 987.2 msl (Dropped to 986.8 after gates opened)

Tailwater at Time of Inspection: 970.7 msl (Rose to 974.0 after gates opened)

Inspection Personnel:

E. A. Nowatzki (CWDD) A. L. Curtis (Town of Wells)

G. S. Salzman (CWDD)

J. Orr (Town of Wells)

E. A. Nowatzki Recorder

Remarks:

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SURFACES: Surface Cracks Spalling	Moderate longitudinal cracks on downstream face of spillway sections. Minor spalling on downstream (REFER TO SHEET 3)	
STRUCTURAL CRACKING	None noticeable	
VERTICAL AND HORIZONTAL ALIGNMENT	Both OK	
MONOLITH JOINTS	Both OK. Joint in left spillway section starting to show wear - minor spalling.	
CONSTRUCTION JOINTS	Starting to show wear.	
RECORDING INSTRUMENTATION	No formal gages. Bar with foot markers upstream of right abutment.	
OTHER	Access to left abutment may not be in public ownership as per local property owner Mr. Nelson.	

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
ANY NOTICEABLE SEEPAGE	None noticeable through spillway sections. Seepage through left embankment (30 ft. earth section behind reinforced concrete wall upstream) and (REFER TO SHEET 3)	This means almost no seepage head loss through left abutment.
JUNCTION OF STRUCTURE WITH Abutment Embankment Other Features	Left spillway to left abutment joint need caulking. Left spillway with gate pier, ditto. (REFER TO SHEET 3)	
DRAINS	Weeps in abutments, both right and left. Drains below spillway monoliths near toe as shown on plans. One observed seeping (4th from left abutment).	
WATER PASSAGES	Refer to "Outlet Works"	
FOUNDATION	Not visible	

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SURFACES: Surface Cracks Spalling	face of spillway sections. Badly spalled on upstream corner of left abutment; reinforcing bar exposed. Crack in concrete face of left abutment upstream, 4' left of left wingwall; shows small bulge upstream.	
ANY NOTICEABLE SEEPAGE	perhaps under and around it. Seepage noticeable through and under left downstream wingwall for distance of several hundred feet downstream. Concrete on wingwall badly spalled even in areas newly patched. 6" diameter pipe	
	through wingwall flowing with 1" of water - steady flow. Seepage over wingwall 6" below spillway crest. Seepage through spall in right downstream wingwall. Weep hole running also. Steel exposed. Crack runs up to top of wall from that point.	
JUNCTION OF STRUCTURE WITH Abutment Embankment Other Features	Right spillway with gate pier ditto. Large spall at crest. Right spillway with right abutment joint need caulking.	

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES ON OUTLET WORKS	Minor erosion on all four piers. Large spall on right side of left pier. Minor spalling on other piers.	Piers are in generally good condition.
INTAKE STRUCTURE	See "EMERGENCY GATE" below.	
OUTLET STRUCTURE	See "EMERGENCY GATE" below.	
OUTLET CHANNEL	Downstream apron and dissipator blocks appear in good condition. One drain in apron was observed to be functioning.	
EMERGENCY GATE	Right gate down for repairs. Left gate opened 4' by operator with electrical portable wrench from gate platform. Power source is on platform. (REFER TO SHEET 2)	Left gate cannot be operated simultaneously with other two gates. Both center and right gates can be operated from right abutment or (REFER TO SHEET 2)

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
EMERGENCY GATE	Middle gate opened $1\frac{1}{2}$" from power source at right abutment. On-off switch at right abutment and on platform. Gates leak when closed. Platform and catwalk in good condition, well maintained, illuminated at night. Rust starting in lift units.	platform; left gate only from platform. Hand wheel also may be used. Emergency generator and hand wheel at Municipal Building.

UNGATED SPILLWAY

Sheet 1 of 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	Generally in good condition. Minor erosion, minor spalls. One large spall at junction with right gate pier.	
APPROACH CHANNEL	None	
DISCHARGE CHANNEL	Apron and dissipation blocks in good condition. Stream has large rock outcrop in center short distance downstream which appears to assist in energy dissipation.	
BRIDGE AND PIERS	Piers for gate platform show signs of minor spalling and erosion (see "OUTLET WORKS").	

INSTRUMENTATION

Sheet 1 of 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	None	
OBSERVATION WELLS	None	
WEIRS	None	
PIEZOMETERS	None	
OTHER	Painted bar upstream of right abutment, with foot marks on it.	

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Variable. Left shoreline well developed (Town of Wells); right shoreline moderately developed. Development on both shorelines along slopes (SEE BELOW)	
SEDIMENTATION	Apparent in photos at inlet; none noticeable at dam.	
SLOPES	approximately 1 vertical to 10 horizontal. Beyond shoreline, hills rise at about 1 vertical to 3 horizontal. Slopes seem stable.	

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION Obstructions Debris Other	Generally in good condition. Rock outcrop short distance downstream. Highway bridge about 1000' downstream, not a serious constriction; right (SEE BELOW)	
SLOPES Cover Stability	Steep; approximately 1:1. Moderately wooded. Rip rap (large stones) immediately downstream from dam. Some erosion on left bank above left wingwall.	
APPROXIMATE NUMBER OF HOMES AND POPULATION	3 homes and 1 tent seen at shore within 1 mile downstream. Bank erosion probably necessary to damage 3 homes. 1 house directly at shore. State-run summer recreation area several miles downstream would be affected. Concur with high hazard.	
CONDITION Obstructions Debris Other	upstream wingwall shows movement. Negligible debris in channel.	

APPENDIX C

COMPUTATIONS

By: EAN Date 8/11/78
Ckd: RDD Date 8/ /78
Subject: Hydraulics of Lake Algonquin Dam

Job # A 78-25-11L
Sheet 1 of 2

FLOW OVER SPILLWAY

Assume maximum pool (El 991.83) and gates full open, (gate sills at El 975.84).

$\therefore H = 991.83 - 975.84 \cong 16$ ft.

From p 373 of BUREC DESIGN OF SMALL DAMS

$$Q = CLH_e^{3/2}$$

$$L = L' - 2(NK_p + K_a)H_e$$

$H_e = H$
$L' = 66'$
$N = 4$
$K_p = 0.01$
$K_a = 0$

$L = 66 - 2(4(0.01) + 0)16$
$L = 66 - 1.28 = 64.72$
$P \cong 7'$
$C = 3.22 + 0.4\left(\frac{H}{P}\right) = 3.22 + 0.4\left(\frac{16}{7}\right) = 4.13$

(use 3.95 as for vertical faced ogee spillway, p 378 of BUREC, DESIGN OF SMALL DAMS)

$\therefore Q = 3.95(64.72)(16)^{3/2} = 16361$ cfs.

Contribution from spillway sections with 5 ft of water flowing over them is 7640 (see sheet 3 of hydrology comps)

$\therefore Q_T = 16361 + 7640 = 22990 \cong 24000$ cfs

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By: EMW Date 8/11/78
Ckd: DDD Date 8/11/78
Subject: Hydraulics of Lake Algonquin Dam

Job # A765-11 E
Sheet 2 of 2

GATE AND SPILLWAY FLOWS

Assume maximum observed pool (El 989.34) and gates full open (gate sills at El. 975.84)

1) Flow through gates

$\therefore H = 989.34 - 975.84 = 13.5'$

$H_e = H$

$C = 3.22 + 0.4\left(\frac{H}{P}\right) = 3.22 + 0.4\left(\frac{13.5}{7}\right) = 3.99$

(Use 3.95 as for vertical faced ogee spillway, p. 378 BUREC Design of Small Dams)

$L = L' - 2(NK_p + K_a)H_e$

$L = 66 - 2(4 \times 0.01 + 0)13.5 = 64.92'$

$Q_g = 3.95(64.92)(13.5)^{3/2} \doteq 12720$ cfs.

2) Flow over spillway

$Q_s = 3.95(88+85)(2.5)^{3/2} \doteq 2700$ cfs

3) Total flow $\doteq$ 15420 cfs

Assume maximum observed pool (E989.34) and gates shut (gate tops at El. 987.84)

H (spillway section) = 2.5'

H (gate section) = 1.5'

$Q_g = 3.95(66)(1.5)^{3/2} = 480$

$Q_s = 2700$

$Q_T = 3180$ cfs.

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BY: PGM 8/10/78 JOB A780J-11E
CHECKED BY: ADD 8/10/78
SUBJECT HYDROLOGY FLOOD ROUTING SHEET 1 OF 8
LAKE ALGONQUIN DAM

TRIANGULAR HYDROGRAPH PARAMETERS

FROM COMPS BY EAV, THE DAM CLASSIFICATION IS HAZARDOUS DUE TO DOWNSTREAM HABITATION

$\therefore$ SDF = PMF

TO DETERMINE THE PMF FOR LAKE ALGONQUIN DRAINAGE BASIN USE DRAINAGE BASIN #46 FROM UPPER HUDSON RIVER HYDROLOGIC FLOOD ROUTING MODELS. Pg. 120, 123, 125, 130

A_1 = DRAINAGE AREA FOR LAKE ALGONQUIN = 263 sq mi. FROM APPLICATION
A_2 = DRAINAGE AREA FOR SUBBASIN #46 = 377 sq mi. N.Y.S. Dam no. 744
$PMF_2 = 2(SPF) = 2(46498) = 92996$ Serial No. 580

$$\left(\frac{A_1}{A_2}\right)^{0.75} = \frac{PMF_1}{PMF_2} \quad ; \quad \left(\frac{263}{377}\right)^{0.75} = \frac{PMF_1}{92996}$$

PMF_1 = 70986 cfs

DETERMINE TIME FOR PEAK INFLOW

T_{p2} = 24 hrs. Pg 125 = TIME OF MAX INFLOW FOR SUBBASIN #46.

$$A_1 = 263 \text{ sq. mi.} = \frac{\pi}{4} d_1^2 \quad ; \quad d_1 = 18.3 \text{ mi}$$

$$A_2 = 377 \text{ sq. mi} = \frac{\pi}{4} d_2^2 \quad ; \quad d_2 = 21.9 \text{ mi.}$$

$$T_{p1} = \frac{d_1}{d_2} T_{p2} = \frac{18.3}{21.9}(24) = 20.1 \text{ hrs.}$$

$$T_b = 2.67^{*} T_{p1} = 2.67(20.1) = 53.5 \text{ hrs.}$$

* FROM DESIGN OF SMALL DAMS Pg 7

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BY: PGM 8/3/78
CHECKED ADD 8/4/78
SUBJECT HYDROLOGY - FLOOD ROUTING
LAKE ALGONQUIN DAM

JOB A7805-11E
SHEET 2 OF 8

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FLOOD STORAGE VS. ELEVATION

LAKE AREA AT POOL ELEV = 280 acres
ASSUME 10:1 SLOPE ON SHORE (from on site observation and verified from USGS QUAD MAPS)
LENGTH OF SHORE LINE = 3.08 mi. (FROM USGS QUAD MAPS)

POOL ELEV. H_c VOL = (ELEV.)(AREA) + $\frac{ELEV^2 10}{2}$ × LENGTH OF SHORE × $\frac{5280 ft}{1 mi}$ × $\frac{1 acre}{43560}$

986.34 0

987.34 1.0 $= 280(1) + \left(\frac{1^2(10)}{2} \times 3.08 \times \frac{5280}{43560}\right)$

282 = 280 + 2

988.34 2.0 $= 280(2) + \left(\frac{2^2 10}{2} \times 3.08 \times \frac{5280}{43560}\right)$

567.5 = 560 + 7.5

989.34 3.0 $= 280(3) + \left(\frac{3^2 10}{2} \times 3.08 \times \frac{5280}{43560}\right)$

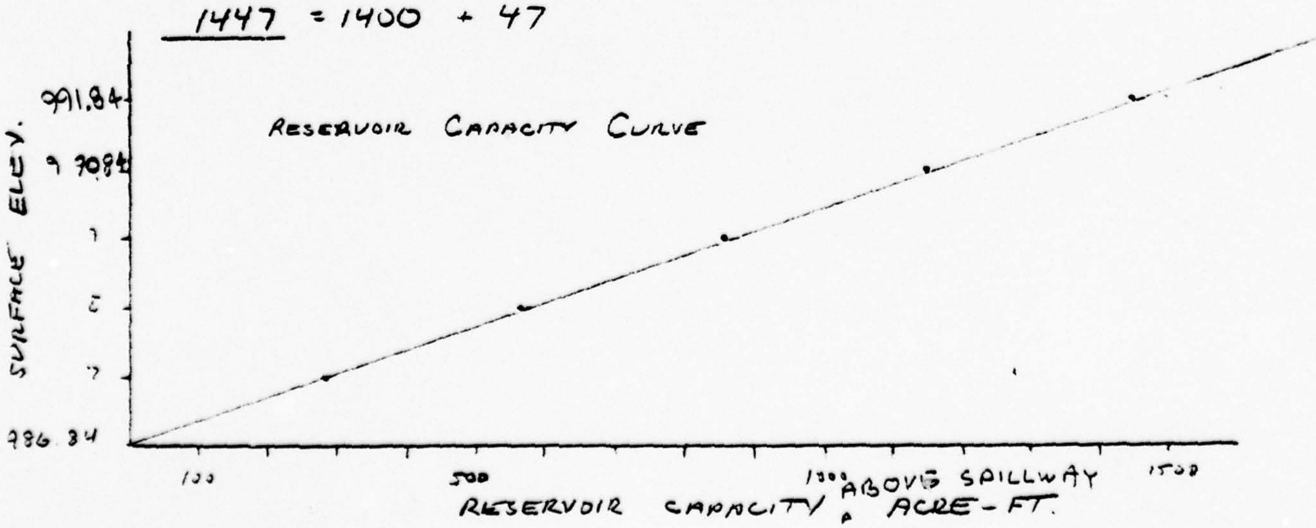
857 = 840 + 17

990.34 4.0 $= 280(4) + \left(\frac{4^2 10}{2} \times 3.08 \times \frac{5280}{43560}\right)$

1150 = 1120 + 30

991.34 5.0 $= 280(5) + \left(\frac{5^2 10}{2} \times 3.08 \times \frac{5280}{43560}\right)$

1447 = 1400 + 47



By: PGM 8/4/78
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SUBJECT: HYDROLOGY - FLOOD ROUTING
LAKE ALGONQUIN DAM

JOB: A780F-11E

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SPILLWAY DISCHARGE CALCULATIONS

$Q = Q_1$ (OVER SPILLWAY) $+ Q_2$ (OVER WEIRS)

$Q_1 = C_{o_1} L H_{e_1}^{3/2}$ FROM DESIGN OF SMALL DAMS Pg. 276

$C: Cd = 3.22 + 0.40\, h/p$ FROM FLUID MECHANICS 7-45

$Q_2 = Cd_2 L_2 H_{e_2}^{3/2}$ WHERE L = EFFECTIVE LENGTH OF CREST FROM DESIGN OF SMALL DAMS Pg. 373

$C_{o_1} = 3.95$ FROM DESIGN OF SMALL DAMS Pg 276
$L_1 = 173'$ FROM DESIGN PLANS OF LAKE ALGONQUIN DAM
H_{e_1} = HEAD OF H_2O OVER SPILLWAY

$Cd_2 = 3.22 + 0.40 \frac{H_e}{P}$ WHERE P = HEIGHT OF WEIRS = 12' FROM DESIGN PLANS.

$L_2 = L' - 2(NK_p + K_a)H_e$ FROM DESIGN OF SMALL DAMS Pg 373

$L' = 66'$ LENGTH OF CREST
$N = 4$ PIERS
$K_p = 0.01$ PIER CONTRACTION COEFFICIENT
$K_a = 0$ ABUTMENT CONTRACTION COEFF.
H_e = TOTAL HEAD ON CREST

FROM DESIGN PLANS.

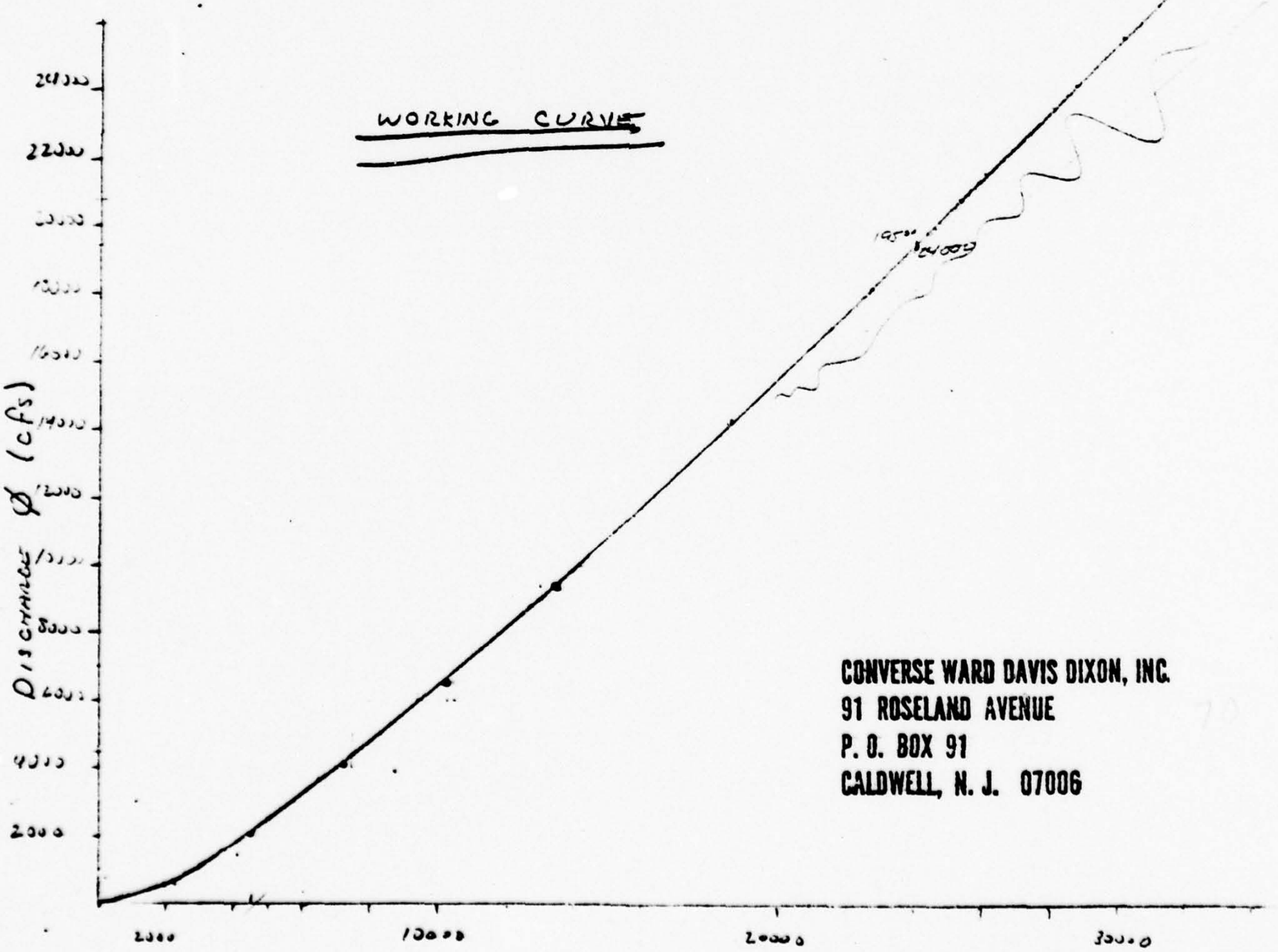
$L_2 = 66 - 2(4 \times 0.01 + 0)H_e$

$H_{e_2} = H_{e_1} - 1$

ELEV.	H_e	Q_T =
987.84	1	$= 3.95(173)(1)^{3/2} + 0$
	<u>683</u>	$= 683$ cfs
988.84	2	$= 3.95(173)\,2^{3/2} + (3.22 + 0.4\frac{1}{12})[66 - 2(4 \times 0.01 + 0)1]\,1^{3/2}$
	<u>2145</u>	$= 1933 + 212$
989.84	3	$= 3.95(173)\,3^{3/2} + [(3.22 + 0.4\frac{2}{12})][66 - 2(4 \times 0.01 + 0)2]\,2^{3/2}$
	<u>4164</u>	$= 3551 + 3.29(65.84)\,2^{3/2} = 3551 + 613$
990.84	4	$= 3.95(173)\,4^{3/2} + [(3.22 + 0.4\frac{3}{12})][66 - 2(4 \times 0.01 + 0)3]\,3^{3/2}$
	<u>6601</u>	$= 5467 + 3.32(65.76)\,3^{3/2} = 5467 + 1134$
991.84	5	$= 3.95(173)\,5^{3/2} + [(3.22 + 0.4\frac{4}{12})][66 - 2(4 \times 0.01 + 0)4]\,4^{3/2}$
	<u>9400</u>	$= 7640 + 3.35(65.68)\,4^{3/2} = 7640 + 1760$

By: PGM 8/4/78 JOB: A7805-11E
CHECKED BY ADD 8/4/78
SUBJECT: HYDROLOGY - FLOOD ROUTING SHEET 4 OF 8
LAKE ALGONQUIN DAM

ELEV (ft)	Ø (cfs)	Ø/2	FLOOD STOR. (ACRE-FT)	FLOOD STOR. (cfs-hrs)	S/ΔT (2 hrs)	SI = Ø/2 + S/ΔT
POOL (986.84)	0	0	0	0	0	0
987.84	683	342	282	3412	1706	2048
988.84	2145	1073	568	6873	3437	4510
989.84	4164	2082	857	10370	5185	7267
990.84	6601	3300	1150	13915	6957	10257
991.84	9400	4700	1447	17509	8755	13455



SI = Ø/2 + S/ΔT

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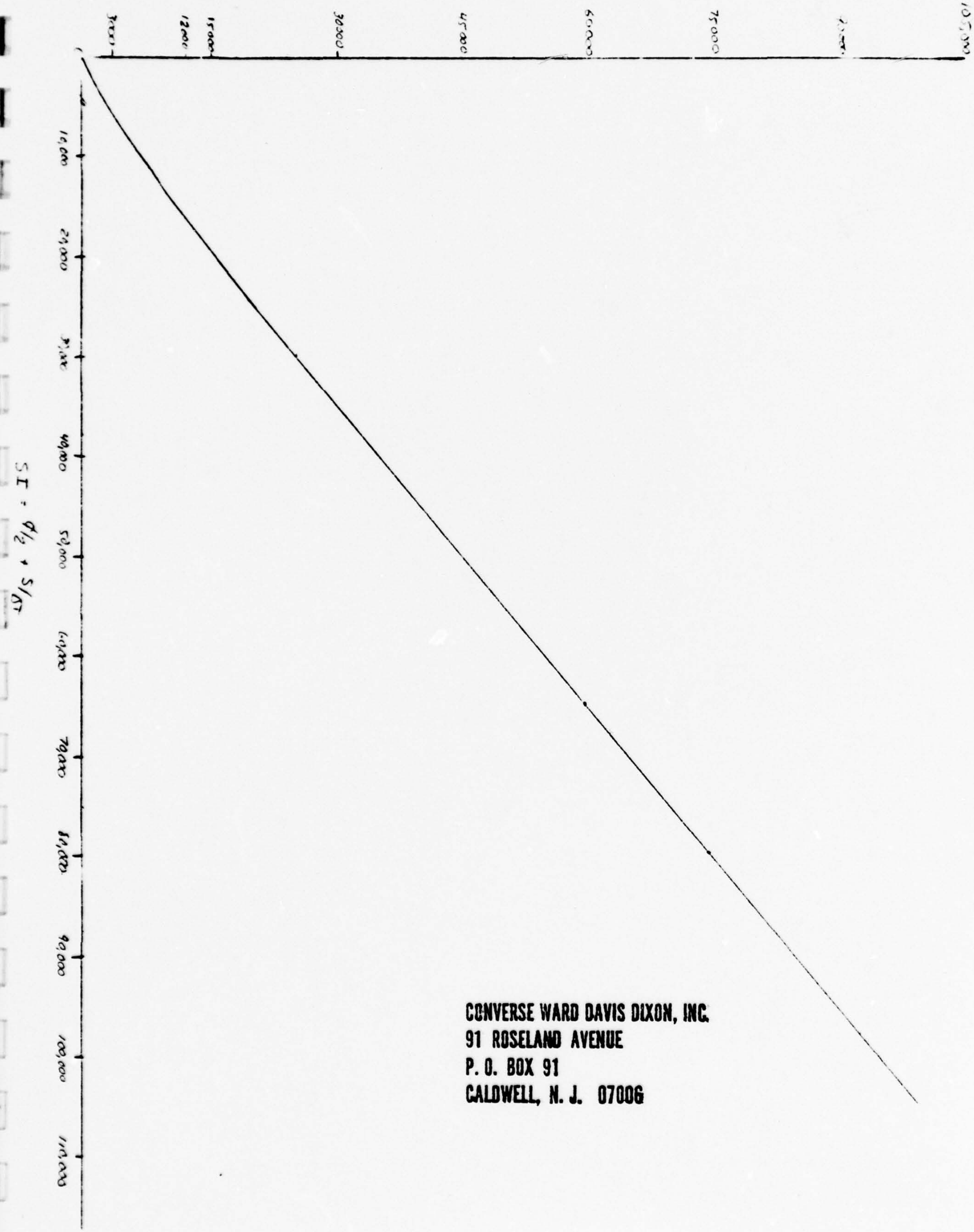
BY ADD 8/7/78 Hydrology - Flood Routing - Lake Algonquin Dam JOB [illegible]

2ND PGM 8/7/78 SHEET 5 OF 8

Discharge Q (cfs)



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SUBJECT: HYDROLOGY - FLOOD ROUTING
LAKE ALGONQUIN DAM

JOB A7805-11E

SHEET 6 of 8

TIME (HRS)	I cfs	Ī cfs	ΣI	Ø
0	0	0	0	0
2.0	7,250	3625	3625	1550
4	14,000	10625	12700	8800
6	21,000	17500	21400	17000
8	27,500	24250	28650	24200
10	34,750	31125	35575	31200
12	42,000	38375	42750	38500
14	48,750	45375	49625	45,500
16	55,750	52250	56375	52,000
18	63,000	59375	63750	59,750
20	70,000	66500	70500	66,500
22	67,500	68750	72750	68,750
24	62,750	65125	69125	65,000
26	58,500	60625	64950	60,700
28	54,500	56500	60550	56,500
30	50,000	52250	56300	52,100
32	46,000	48000	52200	48,000
34	41,500	43750	47950	43,800
36	37,500	39500	43650	39,400
38	33,250	35375	39625	35300
40	28,750	31000	35325	30950
42	24,500	26625	31000	26600
44	20250	22375	26775	22400
46	16250	18250	22625	18200
48	11750	14000	18425	14150
50	7500	9625	13900	9900
52	3250	5375	9375	6000
54	0	1625	5000	2500

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By PGM 8/10/78 — JOB: A7805-11E

CHECKED BY HDD 8/10/78

SUBJECT: HYDROLOGY - FLOOD ROUTING — SHEET 7 of 8

LAKE ALGONQUIN DAM

INFLOW + OUTFLOW HYDROGRAPHS

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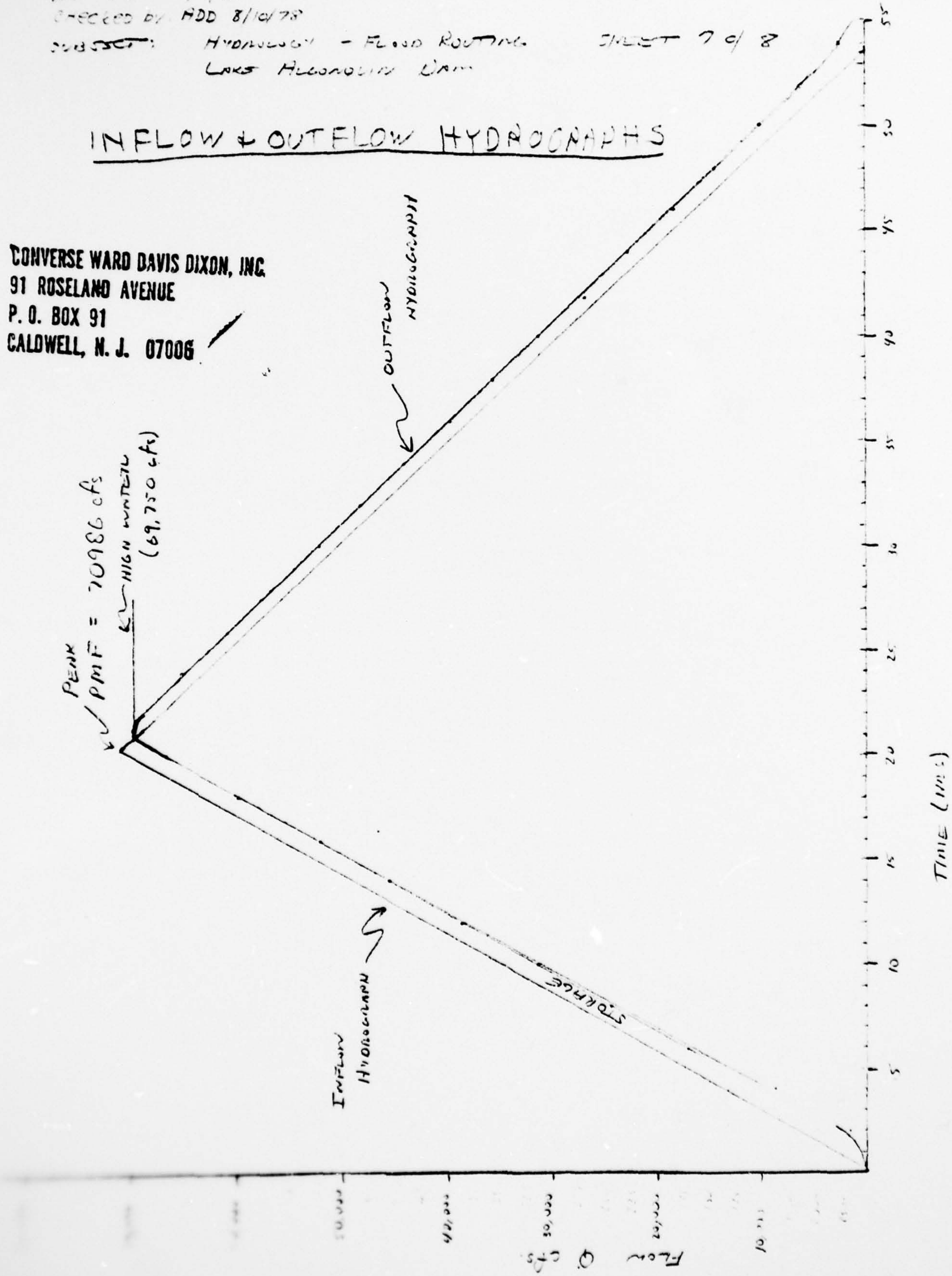


By: PGM 8/10/78
Checked by: ADD 8/10/78
Subject: Hydrology - Flood Routing
Lake Algonquin Dam

JOB: A 7807-11 E

SHEET 8 of 8

TO DETERMINE ELEV. OF THE SURFACE AT PMF

$$Q = C_1\, 173\, H_e^{3/2} + \left[3.22 + 0.4\frac{H_e - 1}{12}\right]\left[66 - 2(4\times 0.1)H_e - 1\right](H_e - 1)^{3/2}$$

ASSUME $H_e = 20'$

$$Q = 59264 + (3.85)(64.48)\,19^{3/2} = 79824$$

ASSUME $H_e = 18'$

$$Q = 50732 + 3.79(64.64)\,17^{3/2} = 67904 \rightarrow 69{,}750 \text{ @ HIGH WATER}$$

∴ PMF WILL RAISE ABOVE ELEV. ± 18' TO ELEV. ±1001±

THIS WILL SERIOUSLY OVERTOP LAKE ALGONQUIN DAM

% OF PMF THAT CAN BE PASSED IS:

$$\frac{9400 \text{ MAX Q THRU SPILLWAY}}{69750 \text{ MAX Q @ HIGH WATER}} \times 100 = 13.5\%$$

CONVERSE WARD DAVIS DIXON, INC.
91 ROSELAND AVENUE
P. O. BOX 91
CALDWELL, N. J. 07006

BY J.K. DATE 7/31/78 **JOSEPH S. WARD + ASSOC** SHEET NO. 1 OF 9

CHKD. BY PG[illegible] DATE 8/2/78 91 ROSELAND AVE. CALDWELL, N. J. JOB NO. A7805-IIE

SUBJECT Stability of Lake Algonquin Dam

Revised 9/19/78 JK

Reference drawings:

Erdman, Anthony and Hosley, Rochester, N.Y

5753-2 spillway cross-section sheet 2 of 8 dated 6/19/5[illegible]

Case 2 more critical → # 5753-3 stability diagrams sheet 3 of 8 dated 6/19/5[illegible]

Checking the stability of 1' length of the dam with water at El. 990.34' as per drawing # 5753-3

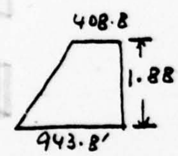
<u>Stability of the section through the dam at El. 984.8[illegible]</u>

Wt of Concrete $= \left(\frac{6+1.9}{2}\right) 1.88 \times 145 = 1078$ lb. ✓

Wt. of water $= \left(\frac{5.5+3.62}{2}\right) \times 1.8 \times 62.4 = 512$ lb. ✓

Total vertical weight $=$ 1590 lb. ✓

Resisting force against sliding $= \mu$ 1590 lb.



Horizontal thrust of Water $= \frac{w}{2}\left(h_1^2 - h_2^2\right) = \frac{62.4}{2}\left[(5.5)^2 - (3.62)^2\right]$

$= 943.8 - 408.8 = 535$ lb.

<u>For stability against sliding</u> $\mu > \frac{535}{1590} = 0.34$ → close to original calculation

Taking $\mu = 0.65$ for concrete on concrete

F.S. $= \frac{0.65 \times 1590}{535} = 1.93$ ∴ ok

Centroid of horizontal water thrust $= \frac{1.88(2 \times 943.8 + 408.8)}{3(943.8 + 408.8)} = 1.06$

Shear stress $= \frac{\text{Horizontal Water Thrust}}{\text{Resisting Area}} = \frac{535}{6 \times 12 \times 12} = 0.62$ p[illegible]

∴ ok ✓

$1.88 - 1.06 = 0.82$

<u>For location of the resultant of forces</u>, taking moments about the toe of the section considered i.e. about pt. A on sheet [illegible]

$-0.82 \times 535 + 3 \times 1078 + 5.4 \times 512 = 1590\,\bar{x}$

$-438.7 + 3234 + 2764.8 =$

or $\bar{x} = \frac{5560.1}{1590} = 3.5$ ft. ✓

which is almost at the center of the base ∴ ok

BY J. K. DATE 9/19/78

CHKD. BY DRB DATE 9/19/78

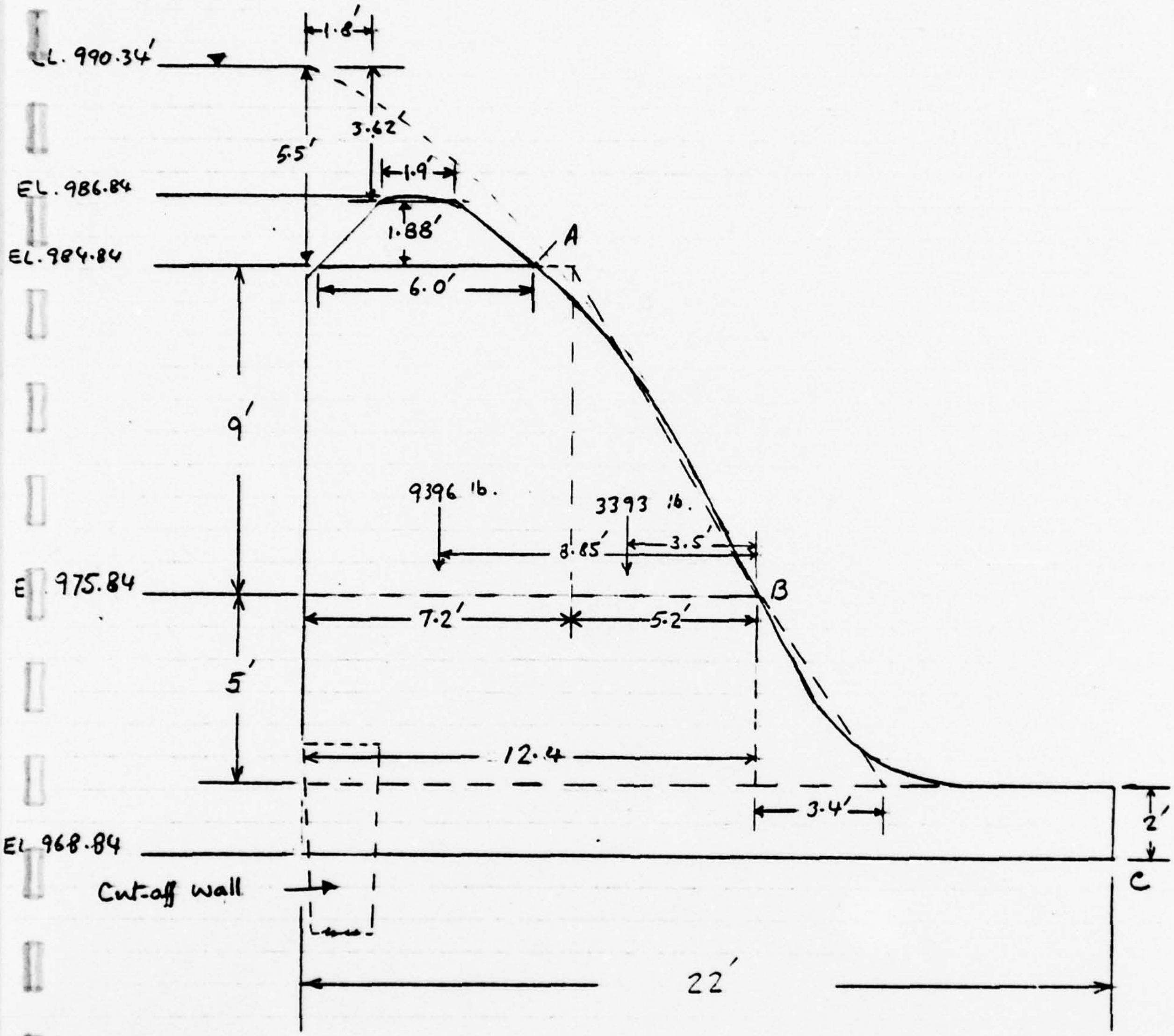
SUBJECT Stability of Lake Algonquin Dam

JOSEPH S. WARD

91 ROSELAND AVE. CALDWELL, N. J.

SHEET NO. 1 A OF 9

JOB NO. A 7805-11E



Applies to computation sheets 1 through 4

Revised 9/19/78 JK

Stability of the section through the dam at El. 975.84

Wt. of Concrete = $1078 + (7.2 \times 9 \times 145) + (\frac{1}{2} \times 9 \times 5.2 \times 145)$
$= 1078 + 9396 + 3393 = 13867$ lb.

Wt. of water + wt. of concrete = $512 + 13867 = 14379$ lb.

Resisting force against sliding = μ 14379 lb.

Horizontal thrust of water = $\frac{\omega}{2}(h_1^2 - h_2^2) = \frac{62.4}{2}\left[(14.5)^2 - (3.62)^2\right]$
$= 6151$ lb.

For stability against sliding μ should be greater than $\frac{6151}{14379} = 0.$

Original calculations requires μ of 0.50.

Taking $\mu = 0.65$

F.S. against sliding $= \frac{0.65 \times 14379}{6151} = 1.5$ ∴ ok.

Shear stress = $\frac{\text{Horizontal Water Thrust}}{\text{Resisting Area}} = \frac{6151}{12.4 \times 12 \times 12} = 3.44$ psi

Very close to original calculated value of 3.46 psi. Both are within safe limits.

For location of the resultant of forces, taking moments about the toe of the section i.e. about pt. B on sheet 1A

$\overset{5836.8}{11.4 \times 512} + \overset{9702}{9 \times 1078} + \overset{83154.6}{8.85 \times 9396} + \overset{11875.5}{3.5 \times 3393} - \frac{10.88}{3} \times 6151 = 14379\ \bar{x}$

or $\bar{x} = \frac{88261}{14379} = 6.14$

Which is close to original calculations.

Base width of the section considered is 12.4. The resultant therefore falls within the middle third and OK against overturning.

BY J. K. DATE 8/1/78 **JOSEPH S. WARD** SHEET NO. 3 OF 9
CHKD. BY PCM DATE 8/2/78 91 ROSELAND AVE. CALDWELL, N. J. JOB NO. A7305-11E
SUBJECT stability of Lake Algonquin Dam

revised 9/19/78 JK

Stability of the dam at its base, El. 968.84

Wt. of Concrete $= 13867 + \left[(5 \times \overset{62.5}{12.4}) + (\tfrac{1}{2} \times 5 \times \overset{8.5}{3.4}) + (22 \times 2)\right] 145$
$= 13867 + 8990 + 1232 + 6380 = 30469$ lb.

Neglect weight of water, being a small fraction and due to addition of cut-off wall weight which should have been subtracted.

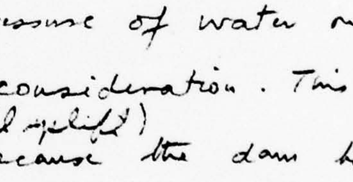
Horizontal thrust of water $= \frac{w}{2}\left(h_1^2 - h_2^2\right) = \frac{62.4}{2}\left[(21.5)^2 - (3.62^2)\right]$

$= 14013$ lb.

For stability against sliding $\mu > \frac{14013}{30469} = 0.46$ ✓

Original calculations as shown on drawing require μ to be greater than 0.94 which is needed only if uplift pressure of water on the base of the dam is taken into consideration. This will be very conservative approach (including full uplift) because the dam has been placed on select gravel and also provided with longitudinal drains. Apron has got weep holes to drain away the water. Also cut off sheet piles and clay blanket upstream of the dam will effectively reduce the uplift pressure.

An approximate but realistic approach would be to assume a triangular distribution of uplift with half the head of water at the U/s end and zero at D/s end.



Uplift pressure $= \frac{w\overset{\frac{21.5}{2} = 10.75}{h} \times 22}{2} = \frac{62.4 \times 10.75 \times 22}{2}$

$= 7379$ lb. ✓

BY J.K. DATE 8/1/78 **JOSEPH S. WARD** SHEET NO. 4 OF 9

CHKD. BY Pbm DATE 8/2/78 91 ROSELAND AVE. CALDWELL, N. J. JOB NO. A 7805-11E

SUBJECT Stability of Lake Algonquin Dam

Revised 9/19/78 JK

Net downward force = 30469 − 7379 = 23090 lb.

$\therefore$ reqd $\mu = \dfrac{14013}{23090} = 0.6$ which is also not close to 0.94 calculated in the original design.

To determine whether uplift pressure exists at the base of the dam, it is recommended that piezometers may be installed in the gravel under the dam.

For the assumed uplift distribution and $\mu = 0.4$

Case 2 $F.S. = \dfrac{23090 \times 0.4}{14013} = 0.66$ ✓ Bureau, design of small dams page

Hence the dam is not safe against sliding ~~[illegible]~~. However, the dam is keyed down by putting dowels in the existing concrete cut off wall located at the upstream face of the dam. The restraint of this keying against sliding will be analyzed later. This probably raises the F.S. above 1 and makes the dam safe. See calculations on sheet 8.

For location of the resultant of forces, taking moments about the toe of the dam i.e. about pt. C on sheet 1A (accounting for the uplift forces)

$$\overset{10700.8}{20.9 \times 512} + \overset{19943}{18.5 \times 1078} + \overset{172416.6}{18.35 \times 9396} + \overset{44109}{13 \times 3393} + \overset{142042}{15.8 \times 8990} +$$

$$\overset{10435}{8.47 \times 1232} + \overset{70180}{11 \times 6380} - \overset{100473.21}{7.17 \times 14013} - \overset{103225.33}{(7379 \times \tfrac{2}{3} \times 22)} = 23090\ \bar{x}$$

or $\bar{x} = \dfrac{261828}{23090} = 11.3'$ ✓ which is within the middle third, hence O.K.

Revised 9/19/78 JK

STABILITY OF THE PIER IN THE SLUICEWAY

Drawing # 5753-2 and

5753-3 case 1 will be checked, being more critical. Checking the stability at the base only instead of at various elevations. One bay of 21.5' length will be considered with water at El. 986.84'.

Vertical load (W) (lb.)		Distance of pt. of appln. of load from toe, ie from pt. l on sheet 5A	Moment about the toe = W×l (ft-lb)
$13 \times 8.5 \times 2.5 \times 145 =$	40056	17.75	710994
$8.5 \times 19.3 \times 2.5 \times 145 =$	59468	17.75	1055557
$\frac{1}{2} \times 19.3 \times 4.13 \times 2.5 \times 145 =$	14447	12.08	174520
$12.67 \times 1.5 \times 2.5 \times 145 =$	6889	15.66	107881
$\frac{1}{2} \times 1.5 \times 4.3 \times 2.5 \times 145 =$	1169	7.90	9235
$2 \times 22 \times 2.5 \times 145 =$	15950	11.0	175450
$\frac{1}{2} \times 1.87 \times 1.06 \times 19 \times 145 =$	2730	20.75	56,648
$3.89 \times 1.87 \times 19 \times 145 =$	20041	21.07	422264
$4.95 \times 2.27 \times 19 \times 145 =$	30957	19.0	588183
$\frac{1}{2} \times 4.95 \times 12.27 \times 19 \times 145 =$	83664	13.77	1152053
$2 \times 22 \times 145 \times 19 =$	121220	11.0	1333420
Total =	396591	Total =	5786205

ie. sec n (first group); thick section (second group)

Horizontal static water thrust $= \frac{wh^2}{2} \times 21.5 = \frac{62.4 \times (16)^2 \times 21.5}{2} = 171725$

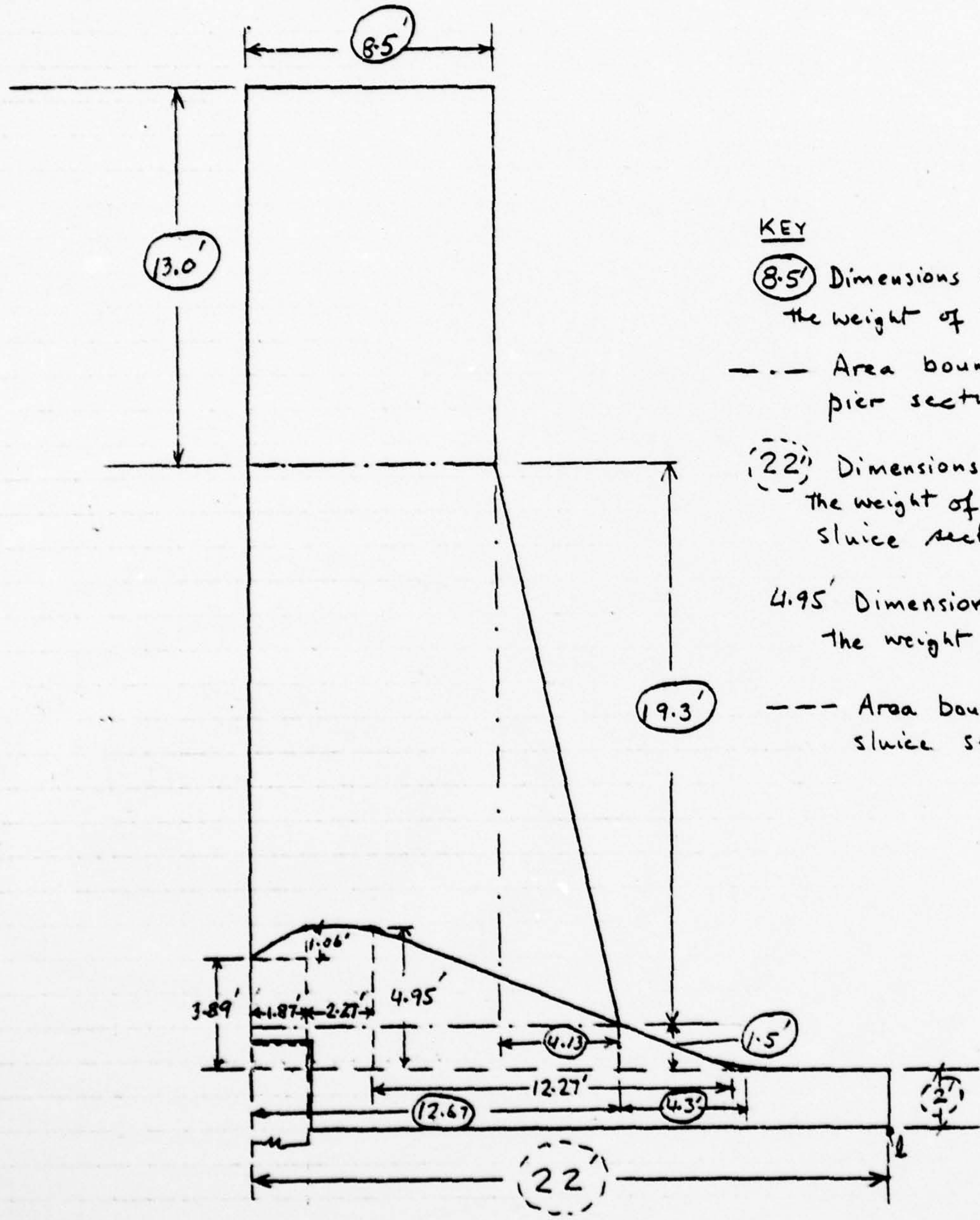
acting at 5.3' from bottom

$h = 986.84 - 968.84 = 18' - 2' = 16'$ (subtracting 2 ft. for thickness of ice)

BY J.K. DATE 9/19/78 — JOSEPH S. WARD — SHEET NO. 5A OF 9

CHKD. BY DRR DATE 9/19/78 — 91 ROSELAND AVE. CALDWELL, N. J. — JOB NO. A 7805-11E

SUBJECT Stability of Lake Algonquin Dam



KEY

8.5' Dimensions used for calculating the weight of the pier section

— · — Area boundaries for the pier section

22' Dimensions used for calculating the weight of both the pier and sluice section

4.95' Dimensions used for calculating the weight of the sluice section

- - - Area boundaries for the sluice section

Applies to computations on sheets 5 through 9

Note: Most dimensions scaled from drawing # 5753-4 by Erdman, Anthony & Hosley, Rochester, N.Y.

Revised 9/19/78 JK

i.e. about pt. l on sheet 5A

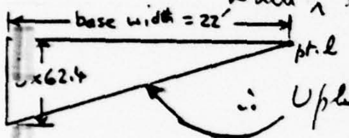
Moment of Water about toe l = 171725 × 5.3 = 910141 ft-lb

Assuming 4 K per lin. ft as ice thrust at el. 936,

total force = 4k × 21.5 = 86000 lb.

Moment about toe, pt. l = 86000 × 15.16 = 1303760 ft-lb.

Assuming triangular uplift distribution with 50% head of water (16 × ½ = 8 ft.) at u/s end and zero at D/s end.



$\therefore$ Uplift $= \frac{wh}{2} \times 22 \times 21.5 = \frac{62.4 \times 8 \times 22 \times 21.5}{2} = 118061$ lb

Moment about toe $= 118061 \times 22 \times \frac{2}{3} = 1731558$ ft-lb

Stability against sliding

For checking the original calculations,

required $\mu = \frac{171725 + 86000}{396591 - 118061}$ (257725) $= 0.93$ which is very close to original computed value on drawing no. 5753-3 (Plate V)

Assuming $\mu = 0.4$ for contact between concrete + gravel

F.S. against sliding $= \frac{(396591 - 118061)\,0.4}{171{,}725 + 86{,}000}$

$= 0.43$ ✓

$\therefore$ it is not safe against sliding. Keying into previously existing concrete cut off wall may have raised the F.S. above 1 and made the dam safe.

Stability against Overturning

$5786205 - 910141 - 1303760 - 1731558 = (396591 - 118061)\,\bar{x}$

$1840746 = 278530\,\bar{x}$

Revised 9/19/78 JK

$$\therefore \bar{x} = \frac{1840746}{278530} = 6.6 \text{ ft.}$$

which is just outside the middle third,

but the weight of the gates and their work platform (concrete footbridge) has not been taken into account. This additional weight will have a stabilizing affect and bring the resultant within the middle third, Therefore, O.K.

Stability against sliding for gravity and pier sections has been re-analyzed in the following pages by taking into account the resistance of the upstream key.

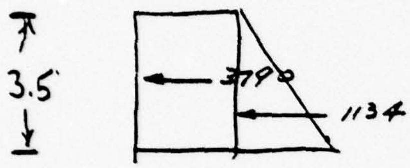
By: EAN Date: 14 Aug 1978 Job # A805-11 E
Chk'd by J.K. Date: 8/14/78 Sheet ~~8~~ 79
Subject: Stability Analysis Lake Algonquin -

Revised 9/19/78 JK

Include effect of cutoff wall for sliding stability
Existing concrete cutoff wall is embedded $3\frac{1}{2}$ feet into foundation soils (sand, inorganic silts, gravel). Assume $K_p = 3.6$

$$\frac{(70\,pcf)(3.5')^2}{2} \times 3.6 \quad + \quad \frac{23090^{\#}}{22\,ft^2} \times 3.6 \times 3.5\,ft$$

$$1544\,\#/ft \quad + \quad 13224\,\#/ft \approx 14770 \quad \#/ft$$



From Sheet 4

$$FS = \frac{23,090 \times 0.4}{14013} = \frac{9,236}{14,013}$$

NOW, NEW $FS = \dfrac{9236 + 14770}{14013} = 1.7$ OK

Case 2

Dowels from gravity section to cutoff wall are #8 bars at 12" centers. Use concrete = 1'9" thick

Assume yield strength of steel = 36,000 psi

Allowable shear stress $= 0.4 \times 36000 = 14400$ psi

$\therefore$ shear strength of dowel $= 14400 \times \frac{\pi}{4}(1)^2 = 11310$ pounds.

Ch'd EAN 8/14/78

Assuming 2000 psi concrete

$\therefore$ allowable shear strength of concrete = 60 psi

Resisting concrete area $= 12 \times 21 = 252$ sq. in.

$\therefore$ Resisting force $= 252 \times 60 = 15120$ pounds.

$\therefore$ steel failure controls.

Case 2

$\therefore F.S. = \dfrac{9236 + 11310}{14013} = 1.5 \quad \therefore$ OK

Ch'd EAN 8/14/78

CONVERSE WARD DAVIS DIXON, INC.
91 ROSELAND AVENUE
P.O. BOX 91
CALDWELL, N.J. 07006

Revised 9/19/78 JK

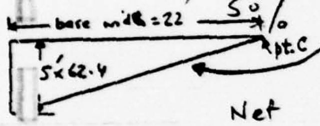
CHECKING STABILITY OF THE DAM WITH WATER AT EL. 991.84 (Ground surface elevation adjoining the abutments)

$\therefore h_1 = 991.84 - 968.84 = 23.0'$ ✓

$h_2 = 991.84 - 986.84 = 5.0'$ ✓

$\therefore$ Horizontal thrust of water $= \frac{w}{2}(h_1^2 - h_2^2) = \frac{62.4}{2}\left[(23)^2 - (5)^2\right] = 15{,}725$ lb ✓

Wt. of concrete from page 3 = 30469 lb. ✓



50% uplift pressure at the base $= \frac{wh \times 22}{2} = \frac{62.4 \times (\frac{23.0}{2}) \times 22}{2} = 7894$.

Net downward force = 30469 − 7894 = 22575 lb. ✓

Passive resistance to upstream cut off wall

$$= \frac{(70\,pcf)(3.5)^2 \times 3.6}{2} + \frac{22575}{22\,ft^2} \times 3.6 \times 3.5$$

$= 1{,}544 + 12{,}929 = 14{,}473$ lb/ft. ✓

Case 3

Hence the shear of dowels still governs (page 3)

$$\therefore F.S. \text{ against sliding} = \frac{(0.4 \times 22575) + 11{,}310}{15{,}725} = 1.3$$

Stability against overturning is not being reevaluated because for $3\frac{1}{2}$ ft. of water over the crest the resultant passed almost through the middle of the base (page 4) and an additional head of $1\frac{1}{2}$ ft. of water over the crest will not alter the overturning stability significantly.

PIER SECTION STABILITY AGAINST SLIDING (ICE THRUST AND KEY CONSIDERED)

Case 1

$$F.S. = \frac{(396591 - 118{,}061)0.4 + (21.5 \times 11{,}310)}{171{,}725 + 86{,}000} = 1.4$$ ✓ (Refer to page 6)

$\therefore$ ok

BY: JK Date 9/19/78 Sheet 9A of 9

Checked by: DRR Date 9.19.78 Job # A7805-11E

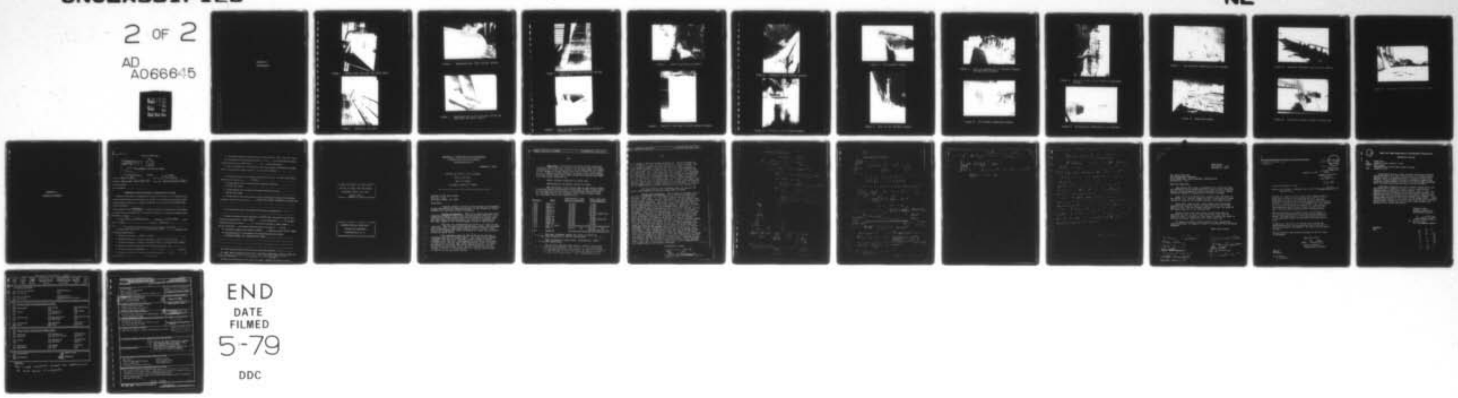
Subject: Summary of Algonquin Dam Stability Analyses

CASE No.	STABILITY	GRAVITY SECTION	GATE/PIER SECTION
CASE 1 WATER AT EL. 986.84' Ice = 4.5K/ft	SLIDING	From original computations on Plate V, it is clear that case 1 is less critical than case 2. Therefore, case 1 has not been analyzed.	F.S. = 1.4 Sheet 9 of 9.
	OVERTURNING		Resultant at the middle third. Sheet 7 of 9.
CASE 2 WATER AT EL. 990.34'	SLIDING	F.S. = 1.5 Sheet 8 of 9.	From original computations on Plate V, it is clear that case 2 is less critical than case 1. Therefore, case 2 has not been analyzed.
	OVERTURNING	Resultant almost at the center of the base. Sheet 4 of 9.	
CASE 3 WATER AT EL. 991.84'	SLIDING	F. S. = 1.3 Sheet 9 of 9.	Not analyzed for reason stated above. A difference of $1\frac{1}{2}$ ft. of water will not create a substantial change.
	OVERTURNING	Additional $1\frac{1}{2}$ ft. of water over case 2 will not substantially change the location of the resultant.	

AD-A066 645 NEW YORK STATE DEPT OF ENVIRONMENTAL CONSERVATION ALBANY F/G 13/2
NATIONAL DAM SAFETY INSPECTION PROGRAM. LAKE ALGONQUIN DAM (NDS--ETC(U)
SEP 78 E A NOWATZKI, G S SALZMAN DACW51-78-C-0035

UNCLASSIFIED NL

2 OF 2
AD A066645



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APPENDIX D

PHOTOGRAPHS

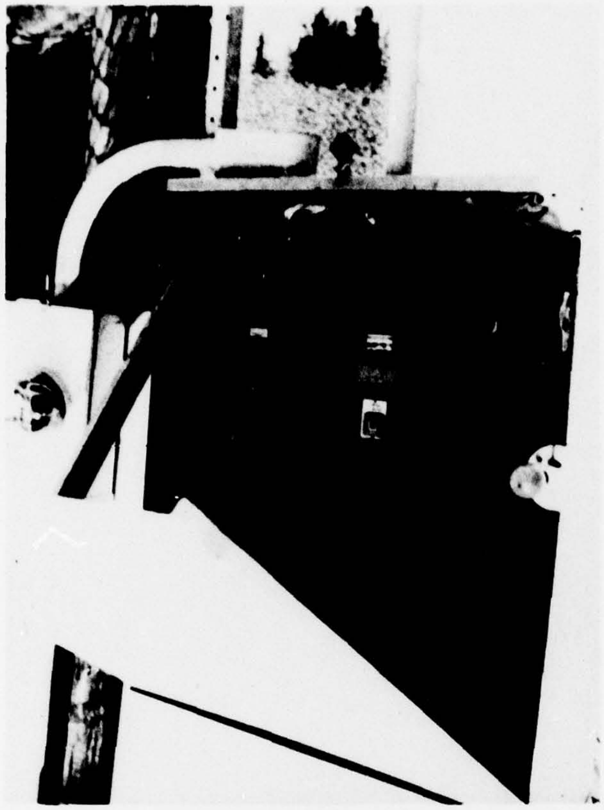


FIGURE 1   CONTROL PANEL FOR RIGHT AND CENTER GATES

FIGURE 2   RAISING OF LEFT GATE

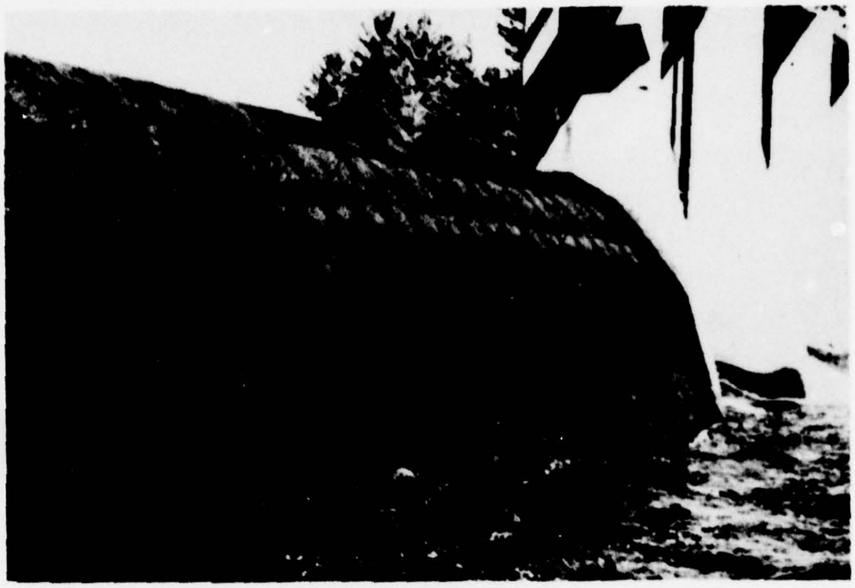


FIGURE 3 DOWNSTREAM FACE, RIGHT SPILLWAY SECTION

FIGURE 4 DOWNSTREAM FACE OF LEFT SPILLWAY SECTION AND RIGHT WALLS OF PIERS 3 AND 4

FIGURE 5 CRACKS IN LEFT GRAVITY/SPILLWAY SECTION NEAR PIER 4

FIGURE 6 SPALL ON RIGHT GRAVITY/SPILLWAY SECTION AT KEY WITH PIER 1

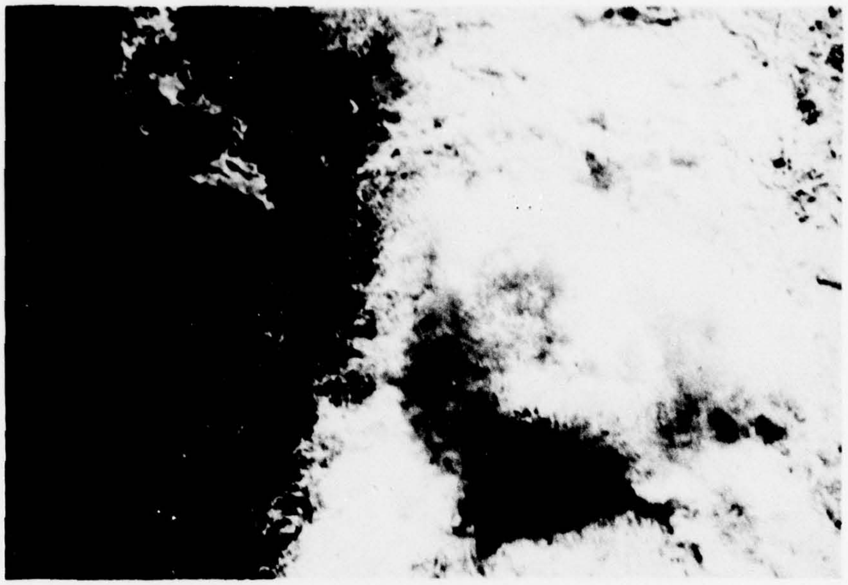


FIGURE 7 ENERGY DISSIPATION BY BAFFLES

FIGURE 8 MAKESHIFT STAFF GAGE AT RIGHT ABUTMENT WINGWALL

FIGURE 9 FEATURES OF RIGHT ABUTMENT SIDEWALL

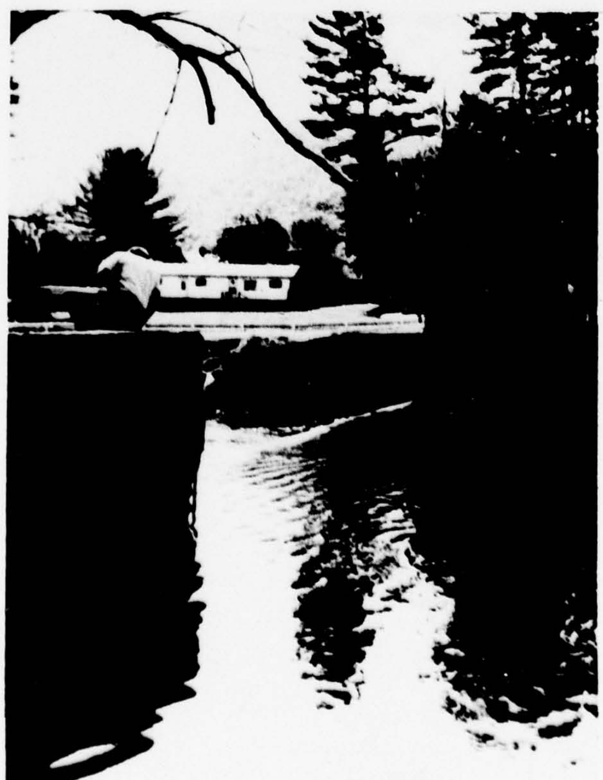


FIGURE 10 FEATURES OF LEFT UPSTREAM WINGWALL

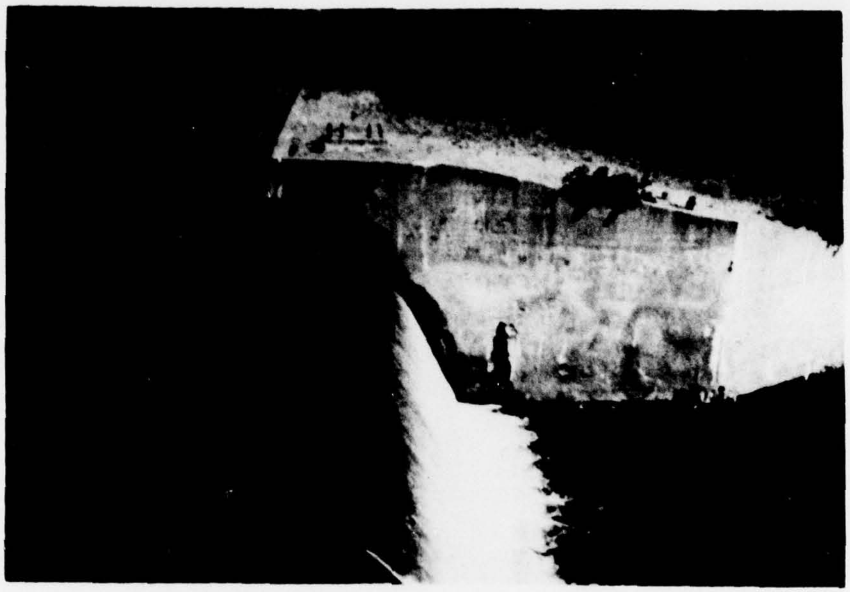


FIGURE 11 LEFT ABUTMENT SIDEWALL

FIGURE 12 SPALL IN LEFT ABUTMENT SIDEWALL

FIGURE 13 SPALL AT JUNCTION OF LEFT ABUTMENT SIDEWALL AND DOWNSTREAM WINGWALL

FIGURE 14 LEFT ABUTMENT DOWNSTREAM WINGWALL

FIGURE 15 DETAILS OF SPALL ON LEFT ABUTMENT DOWNSTREAM WINGWALL

FIGURE 16 RETAINING WALL DOWNSTREAM OF LEFT ABUTMENT

FIGURE 17 FLOW FROM WEEP DOWNSTREAM OF LEFT ABUTMENT

FIGURE 18 DOWNSTREAM CHANNEL

FIGURE 19 TEMPORARY IMPOUNDMENT DAM AT STATE CAMPSITE

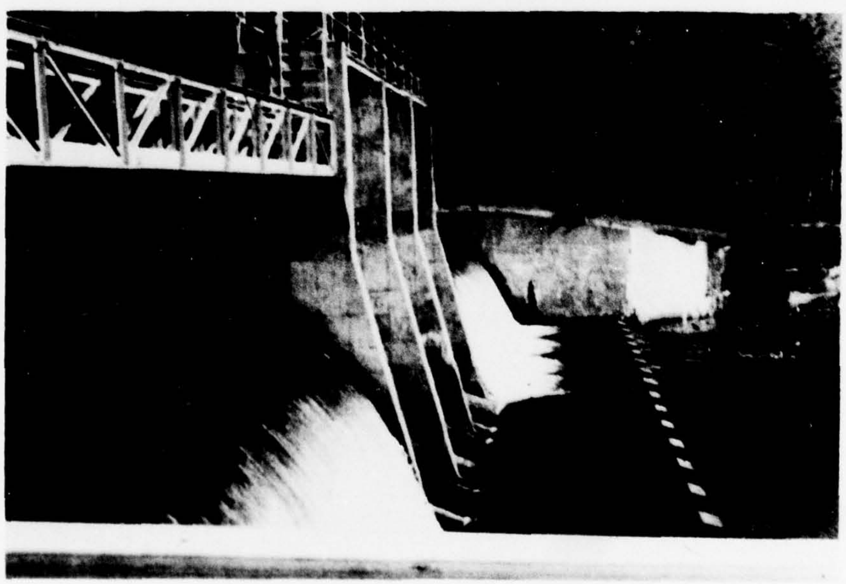


FIGURE 20 FLOW OVER SPILLWAY AT START OF INSPECTION

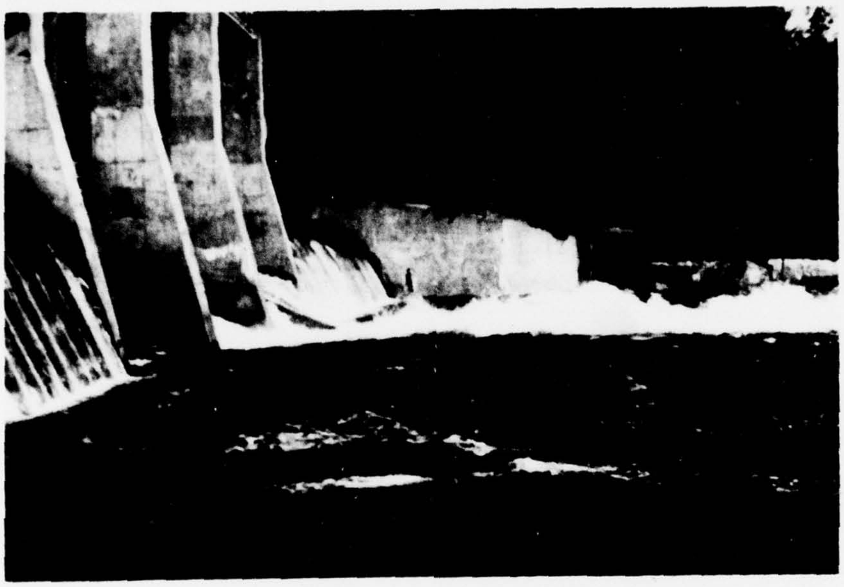


FIGURE 21   TAILWATER 30 MINUTES AFTER OPENING OF GATES

APPENDIX E

RELATED DOCUMENTS

Form E-61. 8-14-52-2M (2D-262)
Ord. 7-10-53

STATE OF NEW YORK

RECEIVED
DEP. CHIEF ENG.
JUL 31 1958
FLOOD CONTROL
CANALS

DEPARTMENT OF PUBLIC WORKS

ALBANY

Received July 31, 1958 Dam No. Orig. #544 171-2700

Disposition Approved August 6, 1958 Watershed Upper Hudson River

Foundation inspected

Structure inspected

Application for the Construction or Reconstruction of a Dam

Application is hereby made to the Superintendent of Public Works, Albany, N. Y., in compliance with the provisions of Section 948 of the Conservation Law (see third page of this application) for the approval of specifications and detailed drawings, marked LAKE ALGONQUIN DAM, eight (8) drawings and specifications

herewith submitted for the ~~construction~~ reconstruction of a dam herein described. All provisions of law will be complied with in the erection of the proposed dam. It is intended to complete the work covered by the application about November 1, 1959
(Date)

1. The dam will be on Sacandaga River flowing into Hudson River in the town of Wells County of Hamilton and
(Give exact distance and direction from a well-known bridge, dam, village, main cross-roads or mouth of a stream)

2. Location of dam is shown on the Lake Pleasant, 1904 quadrangle of the United States Geological Survey.

3. The name of the owner is Town of Wells

4. The address of the owner is Town Supervisor, Town of Wells, N. Y.

5. The dam will be used for Public Recreation and auxiliary water supply

6. Will any part of the dam be built upon or its pond flood any State lands? No

7. The watershed above the proposed dam is 261 sq. mi. square miles.

8. The proposed dam will create a pond area at the spillcrest elevation of 275 acres and will impound 52,528,000 cubic feet of water.

9. The maximum height of the proposed dam above the bed of the stream is 16 feet 11 inches.

10. The lowest part of the natural shore of the pond is 0.5 feet vertically above the spillcrest, and everywhere else the shore will be at least 5 feet above the spillcrest.

11. State if any damage to life or to any buildings, roads or other property could be caused by any possible failure of the proposed dam Some slight damage if dam failed.

12. The natural material of the bed on which the proposed dam will rest is (clay, sand, gravel, boulders, granite, shale, slate, limestone, etc.) Gravel

13. Facing downstream, what is the nature of material composing the right bank? Gravel and loam.

14. Facing downstream, what is the nature of the material composing the left bank? Gravel and loam.

15. State the character of the bed and the banks in respect to the hardness, perviousness, water bearing, effect of exposure to air and to water, uniformity, etc. Gravel and loam, boulders in river bed.

16. Are there any porous seams or fissures beneath the foundation of the proposed dam? - -

17. WASTES. The spillway of the above proposed dam will be 240' ± feet long in the clear; the waters will be held at the right end by a conc. abut. the top of which will be 5 feet above the spillcrest, and have a top width of 1 feet; and at the left end by a conc. abut. the top of which will be 5 feet above the spillcrest, and have a top width of 1 feet.

18. The spillway is designed to safely discharge 17300 cubic feet per second. (Gates open)

19. Pipes, sluice gates, etc., for flood discharge will be provided through the dam as follows: 3 Roller Gates, 12' high and 19' wide

20. What is the maximum height of flash boards which will be used on this dam? - -

21. APRON. Below the proposed dam there will be an apron built of Concrete, 12'-6" wide and 1'-6" thick plus rip-rap 30'-0" wide, plus boulder paving 20'-0". feet long across the stream, feet wide and feet thick.

22. Does this dam constitute any part of a public water supply? Auxiliary water supply.

REPORT ON REPAIR AND REMODELING
OF DAM NO. 544, TOWN OF WELLS,
HAMILTON COUNTY, NEW YORK
August 1949

MORRELL VROOMAN ENGINEERS
Consulting Engineers
Gloversville, N. Y.

MORRELL VROOMAN ENGINEERS
CONSULTING CIVIL ENGINEERS
GLOVERSVILLE, N. Y.

October 7, 1949

REPORT ON REPAIR AND REMODELING

OF DAM NO. 544,

TOWN OF WELLS,

HAMILTON COUNTY, NEW YORK

Members of the Town Board
Town of Wells
Hamilton County, New York

Dear Sirs:

I hereby submit a report on the repair and remodeling of the dam across the Sacandaga River, at the Hamlet of Wells, forming what is known as Lake Algonquin.

General Description. This is a timber crib dam with reinforced concrete wing walls and concrete cut-off wall, located in the southwesterly part of the Hamlet of Wells, about 600 feet above the lower bridge, and 4800 feet southwest of the iron bridge at the northeasterly portion of the Hamlet.

This dam was constructed by the Town of Wells in the year 1924. The dam had 5-foot flashboards, which were fastened with iron hooks so that they could be lowered during winter, and so the flashboards could be lowered or would drop during a period of heavy flood.

The Sacandaga River at the sight of the dam has a watershed area of 263 square miles. The River flows through Sacandaga Lake and Lake Pleasant, located about 14 miles upstream above the dam. The surface area of these two lakes is 3100 acres. Due to the wooded character of the watershed, the nature of the soil, the large annual rainfall, and the storage capacity of the lakes, the dry weather flow of the Sacandaga River is comparatively large, and the Lake formed by the Dam is at all times filled.

Flood Flow. Due to the character of the watershed, the large pondage in the different lakes, the unusually large percentage of wooded area of sand and gravelly soil, the stream is not flashy nor large floods frequent in spite of the steep slopes. Because of the altitude and dense woods, over practically all of the area, the Spring floods are lighter and later than they would otherwise be.

Ample spillway provision has been made.

Nevertheless, occasional floods do occur.

The following is the record made by the United States Geological Survey from actual measurements of the maximum flows at the gauge station near Hope, which includes both the east and west branches of the Sacandaga River, and at which place the watershed area is 491 square miles:

[illegible]ter year	Date	Instantaneous peak flow (second-feet)	Daily peak flow (second-feet)
1934	April 17	10,600	8,650
1935	July 9	11,200	9,040
1936	March 18	23,900	19,400
1937	May 15	9,180	6,800
1938	March 24	16,600	14,100
1939	April 25	11,700	10,400 (April 26)
1940	May 2	10,600	10,200
1941	April 15	11,000	9,850
1942	June 14	14,500	8,570 (June 15)
1943	April 28	10,500	9,670
1944	April 25	10,200	9,020
1945	July 20	20,000	12,600
1946	Oct. 2, 1945	16,700 (unpublished)	12,200 (unpublished)
1947	June 3	16,600 do	11,300 do (11,500 Apr. 12) do
1948	March 22	16,800 (provisional)	13,800 (provisional)

The peak discharge during the flood of March 27, 1913, has been determined as 32,000 second-feet. [illegible]

*The discharge at this point - December 31, 1949 - [illegible] as 30,875 second-feet.

The dam has withstood the floods, shown in the above t[illegible], since its construction, without damage. On December 31, 1[illegible] the flashboards had not been lowered, and had, in addition, [illegible]ced with timbers so that they would not lower without [illegible] of the water against them. Also, 8-inch timbers had

been added to the top of the flashboards. In consequence of this condition, namely the flashboards not being lowered, and being held in place by the timber bracing, which was not a part of the original design of the dam, the water overtopped the wing walls and washed out the roadway at the south of the dam around the wing walls, and took the natural soil, including a house, barn, and other buildings, and washed them downstream. This washout was approximately 150 feet in width, 500 feet in length, and about 14 feet in depth. Little other damage was done to the dam and the timber crib, and masonry wing walls were intact, except some of the timber in the cribs that had been in for 24 years, and had partly decayed, were loosened. A portion of the earth on the upstream side of the dam was washed away.

The dam forms a lake extending over an area of 280 acres, and impounds 52,500,000 cubic feet of water.

Repair and Remodeling. The County of Hamilton refilled the washout area of the roadway and built an earth road thereon. This fill was made from a borrow pit and consists of fine sand, gravel, cobble, and boulders of the natural soil found in this locality. Rock fill was placed at the upper face of the fill adjacent to the pond. Later the existing flashboards were raised, then the pond refilled. There was considerable leakage around the flashboards after they were raised to the maximum height, but there was no leakage or seepage through the fill at the south end of the dam where the main washout occurred. This fill is satisfactory, except that it will have to be raised slightly as contemplated and shown in the plans. It is proposed to repair the decayed or loose timbers in the crib of the dam, to make a stone-fill dam on the natural rock about 70 feet below the main dam, to form a stilling basin. It is also proposed to remove the top course of timbers, and replace these the full length of the dam with reinforced concrete as shown on the plans, and to build an entire new and different type of flashboard that will be lighter, in smaller sections, and can readily be released from each end so that the sections will automatically open with the pressure of the water when the end sections are released, which can be done from the end abutments manually. With the flashboards in place, the spillway section of the dam will have a capacity of 10,000 seconds-feet to the top of the end abutments. With the flashboards released, the overflow capacity will be 26,000 seconds-feet, which will provide for a flood flow of 100 seconds-feet per square mile of the entire watershed.

Respectfully yours,

MORRELL VROOMAN ENGINEERS

By Morrell Vrooman
Morrell Vrooman

MV:M

$62.5 \times 21.5 = 1350$

1.0 ... spacing of counterforts 8'
width ... 14"

$\frac{wl^2}{10} = \frac{1350 \times 8^2}{10} \times 12 = 104000$"#

$1\frac{1}{2}$" ... $p = \frac{...}{... \times 14} = .0033$

$K = .267 \qquad j = 0.91$

$f_s = \frac{M}{p j b d^2} = \frac{104000}{.0033 \times ... \times 12 \times ...} = ...$

Note

The ... is ... and
the ... is ...
by the earth pressure. ...
is ... the ...
pressure.

Overturning $1.08 \times 21.5 = 23.2 \times 1.0 = 23.2$
$2 \times 9 = \frac{18.0}{41.2} \times 4.5 = \frac{81.0}{104.2}$

... $\times 1.0 = 11.5$ ... $= 4.15$ ...

$\frac{...}{73.0}$ ... $\frac{...}{788.9}$

$...$ ... 788

②

Earth on top of footing

$21.5 \times 7.4 = 159'$ $\times 5.4 = 860'$

15,900' 86,000'

Overturning Moment due to earth pressure

$= 19^2 \times \frac{L^3}{6} = 19^2 \times \frac{9^3}{6} = 38,700'$

Overturning M = 38,700 − 86,000 − 21,700 = 69,000'

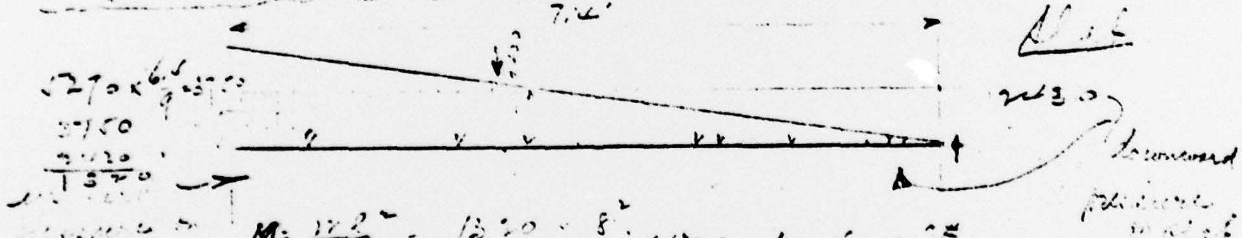
$\frac{69,000}{15,900 + 7250} = 2.96$ OK $\frac{9}{2} = 2'$

23,200

toe pressure $= \frac{23300 \times 1.5 \times 6}{81} + \frac{23250}{9} = 5,270$

5100 − 2600 = 0

Uniform load of earth $\frac{2150}{7.4} = 2,150$ total 2,430



$M = \frac{wl^2}{10} = \frac{1320 \times 8^2}{10} \times 12 = 101,000''\#$

Slab is unreinforced for upward pressure

Tension in concrete $= \frac{101,000 \times 6}{12 \times 22 \times 24} = 87^{\#}/\square''$ (should be reinforced)

$M = \frac{wl^2}{10} = \frac{2430 \times l^2}{10} \times 12 = 187,000''\#$

$f_s = \frac{187,000}{.88 \times 22 \times 4.17} =$

(make up section remains)

Pack of Counterfort [illegible] River

Force of [illegible]

$$\frac{8 \times 38700^{\#}}{10.5} = 30000^{\#}$$

$$2 \text{ Piers } 1''\phi = 2 \times .7854 = 1.5708^{\square''}$$

$$\frac{30000}{1.57} = 19000^{\#}/_{\square''}$$

Item at [illegible] [illegible], [illegible] ..

<u>Memorandum for Mr. McLain.</u>

I have examined the design for the reinforced concrete, counterforted retaining wall shown on "Exhibit G".

I find this design acceptable except that I would change the reinforcement in the bottom slab. As shown, the top is reinforced only at the counterforts while the bottom is reinforced by continuous bars. In order to take care of the distribution of pressure on the slab, I have indicated in a pencil sketch on the blue print an arrangement of bars. I have also indicated in another sketch a detail for the longitudinal bars in the counterfort. This is intended to tie the counterfort into the wall.

W. L. McDowell

Star Route
Northville, N.Y.
January 8, 1978

Mr. Delos Mallette
Acting Regional Forester
N.Y. State Dep't Environmental Conservation
Northville, N.Y.

Dear Mr. Mallette:

Last March,1977 when a tremendous ice jam moved down the Sacandaga River along route 30, Town of Hope, Hamilton County, an unprecedented flood threatened life and caused extensive damage to property and the main highway.

Some of us so affected have learned that the gate of the Wells, N.Y. dam was opened at this time adding a great volume of water to an already swollen, ice-jammed river.

This year, January 9th, heavy rains and moderating temperatures started large ice jams moving in the river. Again at this time an attempt was made to open the gate of the Wells dam but it was frozen shut.

There is yet very high water in the river and an enormous ice jam to move. The next heavy rain will no doubt do this. A repeat of last years disaster may well be avoided by proper regulation of waters above us.

It would be greatly appreciated if your department could implement or suggest a means to alleviate this pressing problem.

Yours very truly,

[illegible] Guthrie
Evelyn Guthrie
Mary Allman
Leo W. Lehmann
Dorothy Lehmann
Aneta Mundell
Harry Mundell

Eugenia W. Call
James Anderson
Stephen Moulton
[illegible]
[illegible]

New York State Department of Environmental Conservation
orthville, New York 12134
18-863-4545

Peter A. A. Berle, Commissioner

RECEIVED JAN 2

DIV. LANDS AND FORESTS REGION 5

ENVIRONMENTAL ANALYSIS UNIT
REGION 5
RAY BROOK, N. Y
JAN 27 1978
DEPT. OF ENVIRONMENTAL CONSERVATION

January 24, 1978

R. Wild / Attention: D. Trost

Subject: Flood Hazard - Sacandaga River, Town of Hope
Hamilton County

Attached is a copy of a letter signed by Mrs. Eugenie Call and others from the Town of Hope regarding the operation of the Wells dam and its effect on downstream flooding. This dam, operated by the Town of Wells, creates Algonquin Lake and effects the flow in the main branch of the Sacandaga River. As I advised you in our recent telephone conversation it is the feeling of these downstream residents that releases are made from this dam at critical times intensifying their flooding problems.

There is presently a very large ice pack in this section of the river which experienced flooding and property damage last spring. This pack created by the floods earlier this month is of considerable concern to these people residing along the river.

In addition to the control of releases from the Wells dam these residents have inquired regarding available sources of aid and assistance in the removal of the threat from the present ice jam.

We will be glad to lend whatever assistance we can to a study of the problem.

Very truly yours,

D. Mallette

Delos H. Mallette
Acting Regional Forester

DHM:plk
Enclosure

cc: T. Monroe
T. D. Shearer

New York State Department of Environmental Conservation

MEMORANDUM

TO: George Koch
FROM: Richard A. Wild by David A. Trost
SUBJECT: Flood Hazard
Sacandaga River, Town of Hope, Hamilton County
DATE: January 30, 1978

In accordance with our recent telephone conversation regarding the hazard created by a dam on Algonquin Lake in the Town of Wells, Hamilton County, I am enclosing a copy of a memo to me from Delos Mallette, the Acting Regional Forester in Northville together with a copy of a letter written to him from one of the parties complaining about the de-watering procedures used by the Town of Wells.

I have sent a letter to Mrs. Call in which I stated that it did not appear that the Department would be able to assist her in a material manner at this point since it appears that, to date, no violations to the Environmental Conservation Law have occurred. The Regional Coordinator for flood and ice jam complaints has been notified and Mrs. Call was informed of the possibility of bringing civil action against the Town of Wells should further damage result. I have forwarded this material to you in case your office has any jurisdiction in this matter. Please feel free to contact either myself or Mr. Mallette for any further information.

Richard A. Wild
Regional Supervisor of
Environmental Analysis

David A. Trost

By: David A. Trost
Sr. Environmental Analyst

RW:DAT:sm
Enc.

NEW YORK STATE
'78 FEB 1 AM 10:25
CONSTRUCTION MANAGEMENT

DWR DAM INSPECTION REPORT wells

RB	CTY	YR AP.	DAM NO.	INS. DATE	USE	TYPE
02	21	58	002700	090970	023	4

AS BUILT INSPECTION

1	Location of Sp'way and outlet	1	Elevations
1	Size of Sp'way and Outlet	1	Geometry of Non-overflow section

1 GENERAL CONDITION OF NON-OVERFLOW SECTION

1	Settlement	1	Cracks	1	Deflections
1	Joints	1	Surface of Concrete	1	Leakage
1	Undermining	1	Settlement of Embankment	1	Crest of Dam
2	Downstream Slope	2	Upstream Slope	1	Toe of Slope

1 GENERAL COND. OF SP'WAY AND OUTLET WORKS

1	Auxiliary Spillway	1	Service or Concrete Sp'way	1	Stilling Basin
1	Joints	1	Surface of Concrete	1	Spillway Toe
1	Mechanical Equipment	1	Plunge Pool	1	Drain

1	Maintenance	C	Hazard Class
3	Evaluation	34	Inspector

COMMENTS:

Dam in good condition except for slight amount of bank erosion at wingwalls

URITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

REPORT NUMBER	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Phase I Inspection Report Lake Algonquin Dam Sacan Daga River Basin, Hamilton County, N.Y. Inventory No. N.Y. 172		5. TYPE OF REPORT & PERIOD COVERED Phase I Inspection Report National Dam Safety Program 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) 10 Edward A. Nowatzki ~~Ph. D.~~ Gary S. Salzman ~~P.E.~~		8. CONTRACT OR GRANT NUMBER(s) 15 DACW51-78-C-0035
9. PERFORMING ORGANIZATION NAME AND ADDRESS Converse Ward Davis Dixon 91 Roseland Avenue, P.O. Box 91 Caldwell, New Jersey 07006		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS New York State Department of Environmental Conservation/ 50 Wolf Road Albany, New York 12233 393 970		11 12. REPORT DATE 27 Sep~~tember~~ 1978 13. NUMBER OF PAGES
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6 National Dam Safety Inspection Program. Lake Algonquin Dam (NDS Number 172), Upper Hudson River Watershed, Sacandaga River, Hamilton County, New York. Phase I Inspection Report.

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Dam Safety
National Dam Safety Program
Visual Inspection
Hydrology, Structural Stability
Hamilton County
Lake Algonquin Dam
Sacan Daga River

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report provides information and analysis on the physical condition of the dam as of the report date. Information and analysis are based on visual inspection of the dam by the performing organization.

Lake Algonquin Dam was judged to be unsafe Non-emergency due to a seriously inadequate spillway.

393 970

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

UNCLASSIFIED

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